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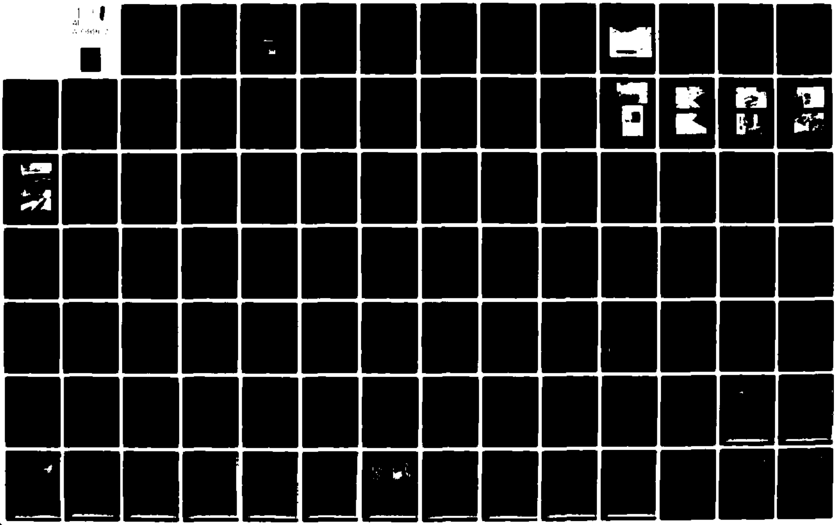

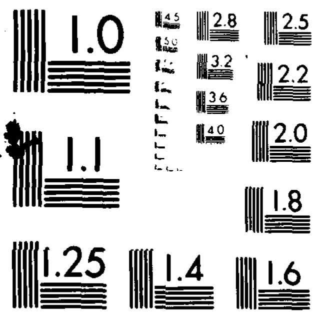

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Sixmile Creek Dam
Oswego River Basin
Tompkins County

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report provides information and analysis on the physical condition of the dam as of the report date. Information and analysis are based on visual inspection of the dam by the performing organization.

Examination of available documents and a visual inspection of the dam did not reveal any conditions which constitute an immediate hazard to human life or property. However, several deficiencies were noted which should be evaluated and remedied.

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The most serious deficiency was a lack of a safe, suitable access to the operating mechanism for the low-level outlet. In the event of an emergency at the dam, it would be extremely difficult to dewater the reservoir under present conditions. The operating condition of the outlet works is also in question.

The spillway could not be examined closely due to lack of access and because discharge was occurring over the weir. A more detailed investigation of the dam and outlet is recommended during a low flow period. The investigation should be commenced within six months of the date of notification of the Owner. The results of such an investigation may indicate the need for a stability analysis of the structure. The analysis, and any remedial measures deemed appropriate as a result of the investigation should be completed within 12 months.

The hydrologic/hydraulic analysis performed indicates that the spillway does not have sufficient capacity to discharge the peak outflow from storms exceeding 7.5 percent of the Probable Maximum Flood (PMF). During a one-half PMF storm, the abutments of the dam would be overtopped by 13 feet. A high tailwater would provide stability, and failure due to erosion is unlikely. If the dam did fail during a one-half PMF storm, the additional flood water would not significantly increase the hazard to downstream areas that would exist prior to failure. Therefore, the spillway is assessed as inadequate.

Since the spillway already extends across the full width of the gorge, there is no way to effectively increase its length or capacity. In addition, it should be noted that even if the dam did not exist, downstream flooding would occur during large storm events due to the configuration of the bedrock channel.

OSWEGO RIVER BASIN

SIXMILE CREEK DAM

TOMPKINS COUNTY, NEW YORK
INVENTORY NO. NY 395

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

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NEWYORK DISTRICT CORPS OF ENGINEERS

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AUGUST 1981

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through frequent inspections can unsafe conditions be detected and only through continued care and maintenance can these conditions be prevented or corrected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, and Spillway Test flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aide in determining the need for more detailed hudrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
SIXMILE CREEK DAM
I.D. NO. NY 395
75A-710
OSWEGO RIVER BASIN
TOMPKINS COUNTY, NEW YORK

A

PHASE I INSPECTION REPORT

NATIONAL DAM SAFETY PROGRAM

Name of Dam:	Sixmile Creek Dam
State Located:	New York
County:	Tompkins
Watershed:	Oswego River Basin
Stream:	Sixmile Creek
Date of Inspection:	July 9, 1981

Assessment

Examination of available documents and a visual inspection of the dam did not reveal any conditions which constitute an immediate hazard to human life or property. However, several deficiencies were noted which should be evaluated and remedied.

The most serious deficiency was a lack of a safe, suitable access to the operating mechanism for the low-level outlet. In the event of an emergency at the dam, it would be extremely difficult to dewater the reservoir under present conditions. The operating condition of the outlet works is also in question.

The spillway could not be examined closely due to lack of access and because discharge was occurring over the weir. A more detailed investigation of the dam and outlet is recommended during a low flow period. The investigation should be commenced within six months of the date of notification of the Owner. The results of such an investigation may indicate the need for a stability analysis of the structure. The analysis, and any remedial measures deemed appropriate as a result of the investigation should be completed within 12 months.

The hydrologic/hydraulic analysis performed indicates that the spillway does not have sufficient capacity to discharge the peak outflow from storms exceeding 7.5 percent of the Probable Maximum Flood (PMF). During a one-half PMF storm, the abutments of the dam would be overtopped by 13 feet. A high tailwater would provide stability, and failure due to erosion is unlikely. If the dam did fail during a one-half PMF storm, the additional flood water would not significantly increase the hazard to downstream areas that would exist prior to failure. Therefore, the spillway is assessed as inadequate.

Since the spillway already extends across the full width of the gorge, there is no way to effectively increase its length or capacity. In addition, it should be noted that even if the dam did not exist, downstream flooding would occur during large storm events due to the configuration of the bedrock channel.

Other deficiencies as outlined below should be corrected within 12 months of the date of notification of the Owner:

1. The operating condition of the sluice gate on the low-level outlet should be tested and repaired as necessary for dependable operation.

2. The deteriorated concrete on the abutments should be repaired, and the brick facing replaced or repaired as necessary.

3. Vegetation growing on the abutment should be removed.

4. The concrete sill on the weir should be repaired to prevent further leakage beneath it.

5. Any open joints in the spillway should be repointed, particularly in the area of heavy efflorescence.

6. An emergency action plan should be developed for the notification and evacuation of downstream residents.

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Approved By:

Col. W. M. Smith, Jr.
New York District Engineer

Date: 22 Sep 81

OVERVIEW
SIXMILE CREEK DAM
NY ID NO. 395

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
SIXMILE CREEK DAM
I.D. NO. NY 395
OSWEGO RIVER BASIN
TOMPKINS COUNTY

SECTION 1: PROJECT INFORMATION

1.1 GENERAL

a. Authority
The Phase I inspection reported herein was authorized by the Department of the Army, New York District, Corps of Engineers, to fulfill the requirements of the National Dam Inspection Act, Public Law 92-367.

b. Purpose of Inspection
This inspection was conducted to evaluate the existing conditions of the dam, to identify deficiencies and hazardous conditions, to determine if these deficiencies constitute hazards to life and property, and to recommend remedial measures where required.

1.2 DESCRIPTION OF PROJECT

a. Description of Dam
The Sixmile Creek Dam is a reinforced concrete arch dam with brick facing. Two outlet conduits and a water supply main pass through the toe near the right (east) abutment of the dam.

The spillway is approximately 65 feet long, constructed as a circular arch with radius of 60 feet in the downstream direction. The upstream face of the spillway slopes at 45 degrees. The maximum height of the spillway is 31 feet from the original stream bed to the crest. The narrow concrete abutments, 9 and 12 wide, stand approximately 5.5 feet above the crest of the weir.

The dam was constructed across the narrow bedrock gorge through which Sixmile Creek flows. The foundation extends to sound rock, approximately 5 to 6 feet, to a maximum 18 feet below the stream bed. Fractures in the bedrock below the foundation were sealed by vertical drill holes filled with compacted clay. The arched weir was constructed using a single course of vitrified shale brick to form the upstream and downstream walls. The brick facing was then reinforced in stages with steel bands set in mortar, before the concrete was added to the interior. The concrete abutments of the dam are also faced with brick.

A 5-foot diameter cast-iron culvert at the toe of the spillway serves as a low-level outlet. Under normal operating conditions, flow through the culvert would be controlled by a sluice gate at the outlet end. The gate control mechanism is visible on the right abutment.

The remains of the intake structure for the water supply system are located in the reservoir, upstream of the dam. It consists of a gatehouse and wet well constructed as a concrete block superstructure on a brick foundation. The intake to the wet well is a rectangular opening with a trash rack, located at the base of the structure. A 24-inch diameter water main extends from the wet well, through the toe of the spillway, and along the downstream channel. An 8-inch diameter blowoff pipe also extends from the wet well to the downstream face of the spillway.

There are no longer any gate control mechanisms inside the abandoned gatehouse. The remains of several rack and pinion mechanisms can be seen on the right abutment of the dam, but their function is unknown.

A low concrete ogee weir spans the stream channel about 500 feet downstream of the dam, creating a large stilling basin in the natural bedrock channel.

b. Location

The Sixmile Creek Dam, known locally as the Thirty-Foot Dam, is located between Routes 79 and 119, approximately 1 mile upstream (southeast) of the City of Ithaca.

c. Size Classification

The dam is a maximum 36.5 feet high at the abutments and has a storage capacity of 287 acre-feet. Therefore, the dam is in the small size category as defined by the "Recommended Guidelines for Safety Inspection of Dams".

d. Hazard Classification

The dam is classified as "high" hazard due to the presence of commercial and residential development adjacent to the stream bed in the City of Ithaca, about 1 mile downstream.

e. Ownership

The dam is owned by the City of Ithaca and formerly operated by the Department of Public Works/Water and Sewer Division. Mr. Philip Cox, City Engineer, was contacted concerning inspection of this dam. His address is: City Hall, 108 Green Street, Ithaca, New York, 14850.

f. Purpose of the Dam

The dam was constructed by the Ithaca Water Co. to create a water supply reservoir. Although no longer in use as a water source, the reservoir was used in the past as a backup supply.

g. Dam and Construction History

The dam was constructed in 1903 for the Ithaca Waterworks Company, and was subsequently taken over by the City of Ithaca for water supply. Plans and construction specifications were prepared by G.S. Williams, who also supervised the construction of the dam by Tucker & Vinton, Inc., of New York City. A detailed discussion of the design and con-

struction history of this dam is available in Paper No. 981 of the Transactions of the American Society of Civil Engineers. The relevant sections of this article have been included in Appendix E.

h. Normal Operation Procedures

There is no normal operating procedure at the dam, and most of the operating equipment is either damaged or missing. The last time the water main was used was in 1959, when the Ithaca Reservoir at Potters Falls Dam was drained to facilitate maintenance work.

1.3 PERTINENT DATA

a. Drainage Area (Sq. mi.)	47
b. Discharge at Dam (cfs)	
Concrete spillway, water surface at top of abutments	2,599
Low-level outlet, water surface at crest of spillway	687
c. Elevation (Plan Datum, approximately equal to USGS Mean Sea Level minus 382)	
Top of Dam	206.5
Crest of Spillway	201
Outlet invert of low-level outlet pipe	170
d. Reservoir Surface Area (acres)	
Top of Dam	20
Crest of Spillway	20
e. Storage Capacity (acre-feet)	
Top of Dam	397
Crest of Spillway	287

f. Dam

Concrete arch spillway and abutments

g. Spillway

Concrete arch, curved weir with concrete abutments;

Length of weir: (ft)		65
Length of abutments (ft):	(east abutment)	12
	(west abutment)	9

h. Low Level Outlets

1. One 60-inch diameter cast-iron culvert with sluice gate at outlet end.
2. One 8-inch diameter blowoff pipe from intake tower.
3. One 24-inch diameter cast-iron water main, no longer functional.

SECTION 2: ENGINEERING DATA

2.1 GEOTECHNICAL DATA

a. Geology

Sixmile Creek Dam is located in the Southern New York Section of the Appalachian Plateau physiographic province. The bedrock in this area consists of shales, siltstones, and sandstones that have been uplifted and gently folded into regional basin structures. The bedrock at the dam is a dark gray, thin-bedded shale with prominent vertical joints that form the steep walls of the gorge below the dam. A review of the "Geologic Map of New York" indicated that there are no faults in the vicinity of the dam.

Surficial soils in the area are the result of glaciations during the Pleistocene Epoch, the last of which was the Wisconsin Glaciation.

b. Subsurface Investigations

There are no records of any subsurface investigation for Sixmile Creek Dam. Continuous bedrock outcrops are visible at the abutments, and in the downstream channel of the dam.

2.2 DESIGN RECORDS

The only engineering data available are included in the ASCE Transactions Paper No. 981 (see Appendix E). No other design records were available.

2.3 CONSTRUCTION RECORDS

The dam was constructed in 1903 by Tucker & Vinton, Inc. There is some correspondence available concerning the construction (see Appendix E). The design engineer, G.S. Williams, supervised the construction. The most significant change in the original design was that the spillway height was reduced from the proposed 90 feet to 30 feet.

2.4 OPERATION RECORDS

No operation records are available for this structure.

2.5 EVALUATION OF DATA

Information used for the preparation of this report was obtained from the Department of Environmental Conservation files and from the City of Ithaca, Department of Public Works/Water and Sewer Division. The information available appeared to be reasonably accurate. A description of the design and construction of the dam written by the design engineer on the project provided the most useful information.

SECTION 3: VISUAL INSPECTION

3.1 FINDINGS

a. General

A visual inspection of Sixmile Creek Dam was conducted on July 9, 1981. The weather was sunny with the temperature in the 80's. The water level at the time of the inspection was slightly above the crest of the weir.

b. Dam

Inspection of the spillway and abutments was hampered by the discharge over the weir, and by the lack of access to the gatehouse, the abutmnets and to the toe of the spillway.

The downstream face of the weir appears to be in fair condition. Some patches of efflorescence could be seen, but for the most part the brick work is intact, with only a few open joints. The crest of the weir was clear of debris. Besides the flow over the crest, water could be seen leaking beneath the concrete sill on the weir.

The concrete and brick abutments are in fair to poor condition. At the right abutment, the brick pavement on the visible downstream corner is eroded, and the underlying exposed concrete is heavily spalled. The remaining brick work on this abutment is in poor condition, with open joints due to missing mortar, and heavy efflorescence. Some vegetation is growing on the abutment at the contact with the bedrock cliff. The left abutment is in similar condition.

c. Low-Level Outlet

The remains of a set of rack and pinion mechanisms are visible on the right abutment of the dam, however, due to lack of access to this area, their condition could not be determined. They did not appear to be operational and their function is unknown.

At the time of the inspection, the low-level outlet pipe was mostly submerged and the sluice gate was apparently closed

The cast-iron stand and gate stem on the right abutment which operates the low-level outlet has reportedly not been used in about 20 years. There is no operator on the gate stand, and the stem which extends to the sluice gate is bent. Again, lack of access to this area prevented a thorough inspection of the equipment.

d. Gatehouse

The gatehouse is in fair condition, with no obvious signs of settlement or displacement of the concrete blocks which form the superstructure. A few open joints were visible between the blocks, and much of the surface is pockmarked by weathering and presumably by vandalism. The brick foundation which was visible above the water line appeared to be in good condition, with the brick work intact.

There is no walkway leading to the gatehouse from either the abutment or the rock cliff above the reservoir. The gatehouse has been abandoned, the door is gone, and no gate valve stems or other operating equipment remain.

e. Downstream Channel

The stilling basin below the dam is generally clear of debris. However, the banks of the channel are very steep and overgrown with trees within this area. Many of the trees have overturned as the banks were eroded, and now overhang the stilling basin.

The 24-inch water main is supported by concrete blocks, and is partially submerged in the stilling basin. The pipe is no longer in use, and due to washouts farther downstream, it is no longer connected to the filtration plant.

The ogee weir which forms the stilling basin is in fair condition. The right wing wall exhibits some spalled concrete at the water line, and a vertical crack down the middle. Some leakage is occurring beneath the wall at the interface with the exposed bedrock surface. The concrete work on the weir itself is in fair condition, with only minor spalling evident along horizontal cracks and vertical construction joints.

Horizontal outcrops of shale occur at the base of the ogee weir and in other areas farther along the stream bed. Just beyond the weir the high, steep, rock walls of the gorge are reduced to low-lying slopes covered with trees.

f. Reservoir Area

The reservoir area is partially contained by the nearly vertical bedrock walls which form the gorge. The remaining area consists of steep, tree-covered slopes. There are no visible signs of instability in the area. However, the steepness of the slopes, and the reported siltation problems known to occur within the Sixmile Creek watershed would indicate that some sedimentation problems may exist in the reservoir area.

3.2 EVALUATION OF OBSERVATIONS

Inspection of the dam revealed the following deficiencies:

1. Lack of access to the spillway, abutments, and gate stand for the low-level outlet, and inoperability of the outlet,
2. Deterioration of the concrete and brick work on both abutments,
3. Vegetation growing on the abutments, and overhanging the stilling basin,
4. Leakage under the sill on the crest of the spillway,
5. Some open jointing, and minor efflorescence on the downstream face of the spillway.

SECTION 4: OPERATION AND MAINTENANCE PROCEDURES

4.1 PROCEDURES

There are no formal operating procedures for this dam, which is no longer used for water supply storage. The only flow occurs as uncontrolled discharge over the spillway.

4.2 MAINTENANCE OF DAM

There is no establisned maintenance plan for this dam. Personnel from the Ithaca Water and Sewer Division reportedly make periodic visits to the site to monitor the condition of the dam.

4.3 WARNING SYSTEM IN EFFECT

No apparent warning system is present for evacuation of downstream residents.

4.4 EVALUTION

The operation procedures in the event of an emergency at this structure are unsatisfactory. In addition, increased maintenance efforts are required to correct the deficiencies noted in Section 3.2.

SECTION 5: HYDROLOGIC/HYDRAULIC

5.1 DRAINAGE AREA CHARACTERISTICS

The 47 square mile (1,498-acre) watershed of Sixmile Creek Dam is indicated approximately on the "Vicinity Map" in Appendix F. Topography in the watershed is generally hilly, with slopes ranging from 5 to 50 percent along the primary drainage path, and from 10 to 35 percent in the uplands. Elevations of the hills which form the drainage divide range from 680 to 1,400 feet above the level of the reservoir.

The watershed is comprised of agricultural and relatively undeveloped open fields and woodlands. Several tracts of State Forest land are included in the eastern part of the drainage area. The Town of Brooktondale located in the southern portion of the watershed is apparently the largest developed area.

The major stream draining the watershed is Sixmile Creek, which for most of its length flows in a steep bedrock gorge. Numerous tributary streams have eroded deep, subparallel channels in the hillsides adjacent to the creek. The somewhat rectangular drainage pattern is typical of areas where drainage has developed on jointed bedrock.

There are few wetland areas within the watershed. The only other significant body of water is Ithaca Reservoir, about 3,000 feet upstream of Sixmile Creek Dam.

5.2 *ANALYSIS CRITERIA*

The analysis of the spillway capacity of the dam and storage of the reservoir was performed using the Corp of Engineers HEC-1 computer model. The unit hydrograph was defined by the Snyder Synthetic Unit Hydrograph method, and the Modified Puls routing procedure was incorporated. The Probable Maximum Precipitation (PMP) was 21.0 inches (24 hrs., 200 sq. miles) from Hydrometeorological Report #33, in accordance with recommended guidelines of the Corps of Engineers. The floods selected for analysis were 50 and 100 percent of the Probable Maximum Flood (PMF) flows. The PMF inflow of 37,350 cfs was routed through the reservoir and the peak outflow was determined to be 37,300 cfs. The one-half PMF inflow was 18,670 cfs and the routed outflow was 18,640 cfs.

5.3 SPILLWAY CAPACITY

The spillway is a 65-foot-long, curved concrete and brick structure which forms an uncontrolled, narrow-crested weir. The abutments of the spillway each stand approximately 5.5 feet higher than the crest. Spillway capacity to the top of the abutments is 2,599 cfs.

5.4 RESERVOIR CAPACITY

The normal water surface is at or near the spillway crest elevation of 201 (plan datum). Using information from a 1925 report by the New York Department of the State Engineer and Surveyor, the impounding capacity

at this elevation is 287 acre-feet (12,500,000 gallons). Surcharge storage capacity to the top of the abutments (El. 206.5) adds 110 acre-feet, which is equivalent to a direct runoff depth of 0.9 inches over the watershed. The total calculated storage capacity is 397 acre-feet. However, due to the heavy siltation rate in the reservoir, the actual capacity may be much lower than the calculated value.

5.5 FLOODS OF RECORD

According to a 1925 report, the maximum flood at the site of the dam was 8,500 cfs in June 21, 1905. No more recent flood records are available.

5.6 OVERTOPPING POTENTIAL

Analyses using the PMF and one-half PMF storm events indicate that the spillway does not have sufficient discharge capacity. The computed depths of overtopping for these two events are 23.2 and 13.0 feet respectively, over the top of the dam. All storm events exceeding 7.5 percent of the PMF will result in the dam being overtopped.

5.7 EVALUATION

The hydrologic/hydraulic analysis indicates that the spillway does not have sufficient capacity to discharge the peak outflow from storms exceeding 7.5 percent of the PMF. However, overtopping of the concrete abutments of the spillway is not likely to cause failure of the dam. Therefore, according to Corps of Engineers guidelines, the spillway should be assessed as *inadequate*.

It should be pointed out that during large storm events flooding would occur in the downstream commercial/residential areas of Ithaca even if the dam did not exist, because the spillway extends across the entire width of the gorge. Since there is no way to increase the length of capacity of the spillway at this site, the Corps criteria for adequacy of the spillway section is not applicable.

SECTION 6: STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations

The dam is a double curvature arch dam constructed of concrete with a facing of mortared brick. The base and abutments tie into bedrock that is horizontally bedded shale.

Visual observation of the spillway and abutments was hampered by discharge over the weir, and by the lack of access to the abutments. Although several localized areas of efflorescence were noted on the face of the spillway, no leakage was apparent through the brick facing.

A wet area was noted in the bedrock wall downstream of the right abutment. This apparent seepage may be the result of groundwater recharging at the top of the wall and moving along fractures and bedding planes in the rock before discharging into the stream channel.

b. Design and Construction Data

A summary of the design and construction history of the dam was prepared by the Engineer on the project, G.S. Williams, and has been included in Appendix E. This was the only design or construction data available.

c. Evaluation of Stability

A stability analysis of an arch dam is beyond the scope of a Phase I inspection. Runoff from one-half the PMF would overtop the dam abutments by 13 feet. However, since the dam is a concrete and masonry structure with bedrock at the abutments, erosion is not likely to cause failure. The high tailwater resulting from spillway discharge and surface runoff downstream will provide stability to the dam during the one-half PMF storm. Furthermore, if the dam were to fail during such a storm, it is unlikely to produce a significant increase in the flooding in downstream areas from that which would exist prior to failure. For these reasons, a stability analysis is not recommended at this time.

It is recommended that the stilling basin be dewatered to permit a more detailed investigation at the toe of the dam under no-flow conditions. The purpose would be to identify any conditions which may affect the stability of the structure. These conditions may include seepage at the toe or through the dam, and/or deterioration of the brick or concrete. The results of such an investigation may indicate the need for a stability analysis of this structure.

d. Seismic Stability

The dam is located in Seismic Zone 1. No seismic stability analysis was performed for this structure.

SECTION 7: ASSESSMENT/RECOMMENDATIONS

7.1 ASSESSMENT

a. Safety
The Phase I Inspection of Sixmile Creek Dam revealed the following deficiencies:

1. Lack of access to the spillway, abutment and operating mechanism for the low-level outlet,
2. Inoperability of the low-level outlet,
3. Deterioration of the concrete and brick work on both abutments of the dam.
4. Vegetation growing on the abutments, and overhanging the stilling basin.
5. Leakage under the sill on the crest of the spillway.
6. Some open jointing, and minor efflorescence, on the downstream face of the spillway.

The spillway capacity is inadequate for the peak outflow from storms exceeding 7.5 percent of the Probable Maximum Flood (PMF). During a one-half PMF storm, the abutments of the dam would be overtopped by 13 feet. However, failure is unlikely to occur because of a high tailwater and the concrete and masonry construction of the dam. Due to the configuration of the gorge, there is no way to increase the length or capacity of the spillway at this site. The spillway is therefore assessed as inadequate.

b. Adequacy of Information
No plans were available for this structure. However, the information provided in the discussion by the design engineer was thorough and appeared to be accurate.

c. Need for Additional Investigations
It is recommended that a more detailed inspection of the spillway be conducted during a period of no flow over the weir. In addition, a close-up inspection of the abutments and low-level outlet is recommended as soon as some suitable access is provided. The results of such an investigation may indicate the need for a stability analysis on the structure.

d. Urgency
The investigation of the spillway section should be undertaken within six months of the date of notification of the Owner. Remedial measures deemed appropriate as a result of the investigation, and other deficiencies as outlined below should be corrected within 12 months of the date of notification.

7.2 RECOMMENDED MEASURES

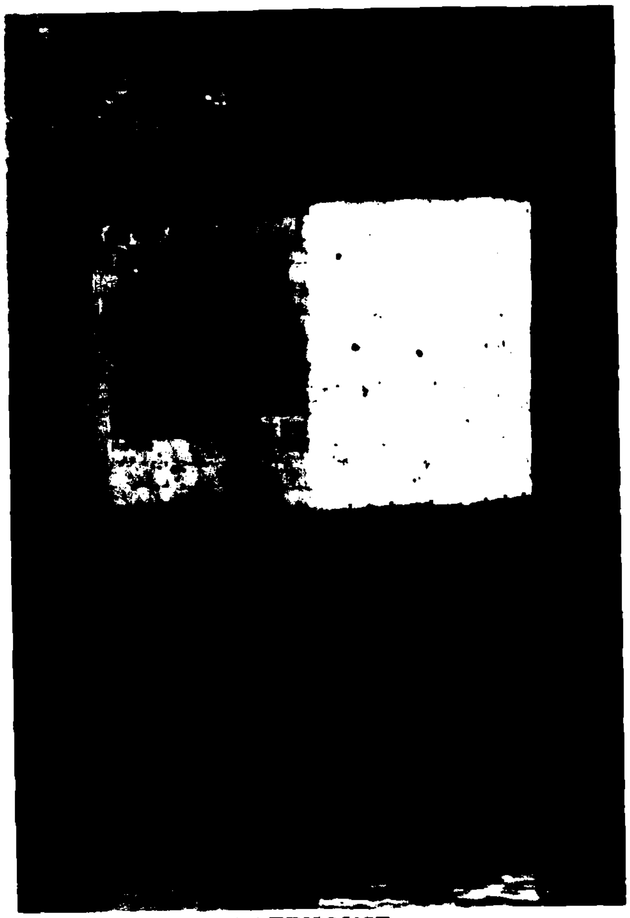
a. Some suitable, safe access should be constructed to the abutments of the dam, particularly to the mechanism on the right abutment which operates the low-level outlet.

b. The operating condition of the sluice gate on the low-level outlet should be tested and repaired as necessary for dependable operation.
c. The deteriorated concrete on the abutments should be repaired, and the brick facing replaced or repaired as necessary.
d. Vegetation growing on the abutments should be removed.
e. The concrete sill on the weir should be repaired to prevent further leakage through it.
f. Any open joints in the spillway should be repointed, particularly in the area of heavy efflorescence.
g. An emergency action plan should be developed for the notification and evacuation of downstream residents.

APPENDIX A

PHOTOGRAPHS

DOWNSTREAM VIEW OF SPILLWAY AND ABUTMENTS



GATEHOUSE

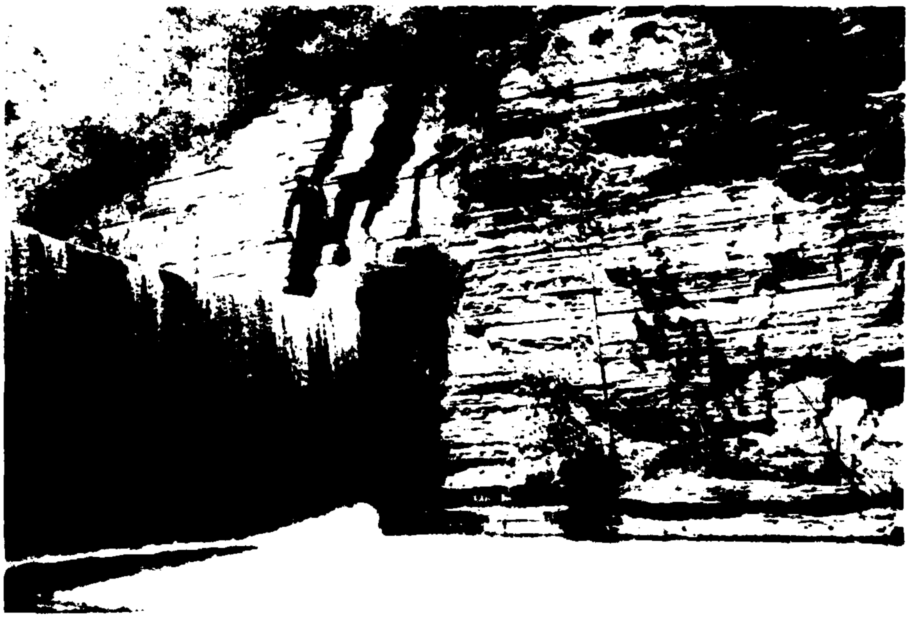
OVERVIEW OF SPILLWAY, RIGHT ABUTMENT, AND GATEHOUSE (NOTE WATER MAIN AT TOE OF DAM)

LEAKAGE UNDER SILL OF SPILLWAY

RACK AND PINION MECHANISM ON LOW-LEVEL OUTLET
(NOTE SPALLING AND OPEN JOINTS ON ABUTMENT)

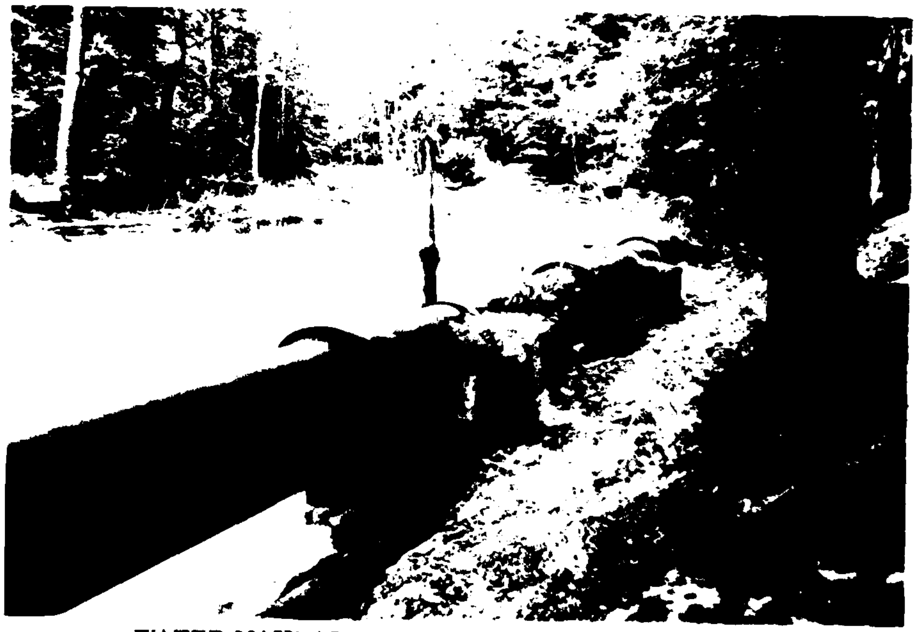
LOW-LEVEL OUTLETS AND WATER MAIN AT TOE OF DAM

UPSTREAM VIEW OF LEFT ABUTMENT



DOWNSTREAM VIEW OF LEFT ABUTMENT

WEIR DOWNSTREAM OF DAM



WATER MAIN ALONG DOWNSTREAM CHANNEL

APPENDIX B

VISUAL INSPECTION CHECKLIST

93-15-3(9/80)

VISUAL INSPECTION CHECKLIST

1) Basic Data

a. General

Name of Dam Sixmile Creek Dam

Fed. I.D. # NY 395 DEC Dam No. 75A-710

River Basin Oswego

Location: Town Ithaca County Tompkins

Stream Name Sixmile Creek

Tributary of Cayuga Inlet

Latitude (N) 42° 25.5' Longitude (W) 76° 28.5'

Type of Dam concrete arch spillway and abutments

Hazard Category C - High Hazard

Date(s) of Inspection July 9, 1981

Weather Conditions sunny, 85°

Reservoir Level at Time of Inspection 201 (Plan datum)*

b. Inspection Personnel Carol Sweet, Reginald Barron, Susan Pierce, William Checchi

c. Persons Contacted (Including Address & Phone No.)

Mr. Philip Cox, City Engineer
City Hall, 108 Green street
Ithaca, New York 14850
607/272-1716

d. History:

Date Constructed 1903 Date(s) Reconstructed –

–

Designer G.S. Williams

Constructed By Tucker + Vinton, Inc., New York

Owner City of Ithaca

* Plan datum ≈ MSL minus 382

2) Embankment

a. Characteristics

(1) Embankment Material no embankment. concrete abutments keyed into bedrock walls

(2) Cutoff Type none

(3) Impervious Core -

(4) Internal Drainage System -

(5) Miscellaneous -

b. Crest N/A

(1) Vertical Alignment

(2) Horizontal Alignment

(3) Surface Cracks

(4) Miscellaneous

c. Upstream Slope N/A

(1) Slope (Estimate) (V:H)

(2) Undesirable Growth or Debris, Animal Burrows

(3) Sloughing, Subsidence or Depressions

(4) Slope Protection ______

(5) Surface Cracks or Movement at Toe ______

d. Downstream Slope N/A

(1) Slope (Estimate - V:H) ______

(2) Undesirable Growth or Debris, Animal Burrows ______

(3) Sloughing, Subsidence or Depressions ______

(4) Surface Cracks or Movement at Toe ______

(5) Seepage ______

(6) External Drainage System (Ditches, Trenches; Blanket) ______

(7) Condition Around Outlet Structure ______

(8) Seepage Beyond Toe ______

e. Abutments - Embankment Contact

Concrete abutments keyed into bedrock walls of gorge.
Bedrock is horizontally - bedded shales/siltstones/sandstones
with vertical and horizontal fracture planes

93-15-3(9/80)

(1) Erosion at Contact none visible

(2) Seepage Along Contact slight seepage through bedrock noted downstream of right abutment. wet stains on rock, slight growth of weeds

3) Drainage System

a. Description of System None

b. Condition of System

c. Discharge from Drainage System

4) Instrumentation (Momumentation/Surveys, Observation Wells, Weirs, Piezometers, Etc.) None

93-15-3(9/80)

5) Reservoir

a. Slopes Very steep-sided bedrock gorge spanned by arch spillway. Reservoir area surrounded by moderate to steep, wooded slopes

b. Sedimentation heavy silt accumulation reported in reservoir

c. Unusual Conditions Which Affect Dam Potters Falls Dam, located approx. 1 mile upstream, also regulates flow into reservoir

6) Area Downstream of Dam

a. Downstream Hazard (No. of Homes, Highways, etc.) Downtown Ithaca situated less than 2 miles downstream

b. Seepage, Unusual Growth none visible - submerged

c. Evidence of Movement Beyond Toe of Dam none visible - submerged

d. Condition of Downstream Channel few fallen trees overhanging stilling basin; otherwise clear of debris.

7) Spillway(s) (Including Discharge Conveyance Channel)

concrete arch spillway with brick paving

a. General uncontrolled spillway discharge into stream channel. Stilling basin at toe of dam created by low concrete ogee weir about 500 feet downstream.

b. Condition of Service Spillway fair; some localized patches of efflorescence, but brick facing intact. Most joints filled with mortar. Concrete sill on crest of weir shows transverse cracks at regular spacing (construction joints?) Flow over crest, and some leakage under sill. Crest clear of debris

c. Condition of Auxiliary Spillway N/A

d. Condition of Discharge Conveyance Channel flat-lying bedrock outcrops visible in channel below ogee weir. Channel clear of debris. Steep bedrock slopes give way to low-lying, vegetated banks.

8) Reservoir Drain/Outlet

Type: Pipe ✓ Conduit ___ Other ___

Material: Concrete ___ Metal cast-iron Other ___

Size: 60-inch diameter Length approx 20 feet

Invert Elevations: Entrance approx 170* Exit approx. 170*

Physical Condition (Describe): Unobservable ✓

Material: cast iron

Joints: (90% submerged) Alignment ___

Structural Integrity: ___

Hydraulic Capability: currently inoperable; calculated capacity is 687 cfs

Means of Control: Gate ✓ Valve ___ Uncontrolled ___

Operation: Operable ___ Inoperable ✓ Other ___

Present Condition (Describe): sluice gate closed; gate mechanism inaccessible, but reportedly inoperable. Located on right abutment

* plan datum, equals MSL minus 382.

9) Structural

a. Concrete Surfaces Concrete exposed where brick paving has eroded from abutments. Heavy spalling, efflorescence.

b. Structural Cracking none visible - lack of access prevented more thorough inspection of abutments

c. Movement - Horizontal & Vertical Alignment (Settlement) none visible

d. Junctions with Abutments or Embankments apparent seepage under right abutment at level of spillway crest

e. Drains - Foundation, Joint, Face N/A

f. Water Passages, Conduits, Sluices sluice gate on low-level outlet submerged; gate stem bent, inoperable
24-inch water main and 8" blowoff pipe visible at toe of spillway - no longer functional

g. Seepage or Leakage leakage under sill on crest of spillway, also under right abutment at junction with spillway crest.

h. Joints - Construction, etc. few open joints in brick paving on downstream face of spillway. Brickwork on abutments is poor, with many missing bricks, open joints, efflorescence from mortar

i. Foundation reportedly founded on bedrock; longitudinal fractures sealed with compacted clay.

j. Abutments poor condition due to spalled and eroded concrete and bricks, vegetation growing on top

k. Control Gates gate on low-level outlet - inaccessible and reportedly inoperable.

l. Approach & Outlet Channels approach channel reportedly silted up. Outlet channel clear of debris. Many trees overhanging banks

m. Energy Dissipators (Plunge Pool, etc.) stilling basin formed in natural stream channel by low ogee weir situated about 500 feet downstream of dam. Some erosion evident on banks of basin

n. Intake Structures Gate house with wet well in reservoir. Fair structural condition, however, water supply facilities have been abandoned and therefore neglected. No operable equipment inside.

o. Stability appears to be stable.

p. Miscellaneous -

10) Appurtenant Structures (Power House, Lock, Gatehouse, Other)

a. Description and Condition Abandoned gate house in reservoir. No service bridge, no door, no operating mechanisms inside. Valves regulating flow through blow off pipe and 24" water main presumably left in closed position. Structural condition of gate house is fair. Concrete superstructure shows few open joints, pock marks; no obvious cracking, displacement, or settlement. Brick foundation intact, good alignment.

24-inch main washed out downstream. No longer connected to filtration plant.

11) Operation Procedures (Lake Level Regulation):

None - flow over weir is uncontrolled, low-level outlet is closed on downstream side of dam.

APPENDIX C

HYDROLOGIC/HYDRAULIC ENGINEERING DATA AND COMPUTATIONS

CHECK LIST FOR DAMS
HYDROLOGIC AND HYDRAULIC
ENGINEERING DATA

AREA-CAPACITY DATA:

	Elevation * (ft.)	Surface Area (acres)	Storage Capacity (acre-ft.)
1) Top of Dam	206.5	20	397
2) Design High Water (Max. Design Pool)	201.0	20	287
3) Auxiliary Spillway Crest	–	–	–
4) Pool Level with Flashboards	–	–	–
5) Service Spillway Crest	201.0	20	287

* Plan datum, approx. equal to MSL minus 382

DISCHARGES

	Volume (cfs)
1) Average Daily	N/A
2) Spillway @ Maximum High Water	2599 cfs
3) Spillway @ Design High Water	N/A
4) Spillway @ Auxiliary Spillway Crest Elevation	–
5) Low Level Outlet – presently inoperable	687 cfs
6) Total (of all facilities) @ Maximum High Water	3286 cfs
7) Maximum Known Flood	8500 cfs (1905)
8) At Time of Inspection	± 20 cfs

CREST: ELEVATION: 206.5 plan datum

Type: concrete arch weir with abutments

Width: – Length: 86 feet

Spillover uncontrolled weir across gorge

Location

SPILLWAY:

SERVICE		AUXILIARY
201.0 plan datum	Elevation	NONE
arch	Type	–
65 feet	Width	–
	Type of Control	
✓	Uncontrolled	–
	Controlled:	
–	Type (Flashboards; gate)	–
–	Number	–
–	Size/Length	–
	Invert Material	–
	Anticipated Length of operating service	–
30 feet	Chute Length	–
unknown	Height Between Spillway Crest & Approach Channel Invert (Weir Flow)	–

HYDROMETEROLOGICAL GAGES:

Type : NONE

Location:

Records:

Date -

Max. Reading -

FLOOD WATER CONTROL SYSTEM:

Warning System: NONE

Method of Controlled Releases (mechanisms):

NONE OPERABLE

DRAINAGE AREA: 47 square miles

DRAINAGE BASIN RUNOFF CHARACTERISTICS:

Land Use - Type: rural - wooded / agricultural

Terrain - Relief: moderate to steep slopes, numerous tributary streams

Surface - Soil: glacial deposits

Runoff Potential (existing or planned extensive alterations to existing (surface or subsurface conditions)

steep slopes; relatively undeveloped area as yet

Potential Sedimentation problem areas (natural or man-made; present or future)

possible siltation problems based on past performance

Potential Backwater problem areas for levels at maximum storage capacity including surcharge storage:

storage capacity based on constant surface area of 20 acres

Dikes - Floodwalls (overflow & non-overflow) - Low reaches along the Reservoir perimeter:

Location: none

Elevation:

Reservoir:

Length @ Maximum Pool 0.3 (Miles)

Length of Shoreline (@ Spillway Crest) 0.8 (Miles)

NONREPRODUCIBLE GRID FORM 145

METCALF & EDDY, ENGINEERS

Project NY C of E DAM INSPECTION Acct No. J 7594 Page 1 of 3

Subject SIX MILE CREEK DAM Comptd By M NOWAK Date 8-10-81

Detail Ck'd By Date

DRAINAGE AREA = 47 sq mi

IMPERVIOUS AREAS = water surface of

Six Mile Creek reservoir :	20 acres
Ithaca reservoir :	47 acres
Total	67 acres

The ratio of impervious area to total area is:

$$\frac{67 \text{ acres} / 640 \text{ acres/sq mi}}{47 \text{ sq mi}} = .0022$$

WATERSHED PARAMETERS

Snyder Unit Hydrograph

Lag time:

$$t_p = C_t (L L_c)^{0.3}$$

where $C_t = 2.0$

$L = 16.4$ mi

$L_c = 6.6$ mi

$$t_p = 2.0 \{(16.4)(6.6)\}^{0.3} = 8.15 \text{ hrs.}$$

Unit rainfall duration

$$t_r = t_p / 5.5$$

$$= 8.15 / 5.5 = 1.48 \text{ hrs}$$

adjusted t_R = 1.0 hr

Adjusted Lag Time

$$t_{pR} = t_p + 0.25 (t_R - t_r)$$

$$= 8.15 + 0.25 (1.0 - 1.48) = 8.03 \text{ hrs.}$$

Peaking Coefficient, $C_p = 0.6$

Project NY G&E DAM INSP — Acct. No. J 7594 — Page 2 of 3
Subject SIX MILE CREEK DAM — Comptd. By M. NOVAK — Date 8-19-81
Detail — Ck'd. By — Date

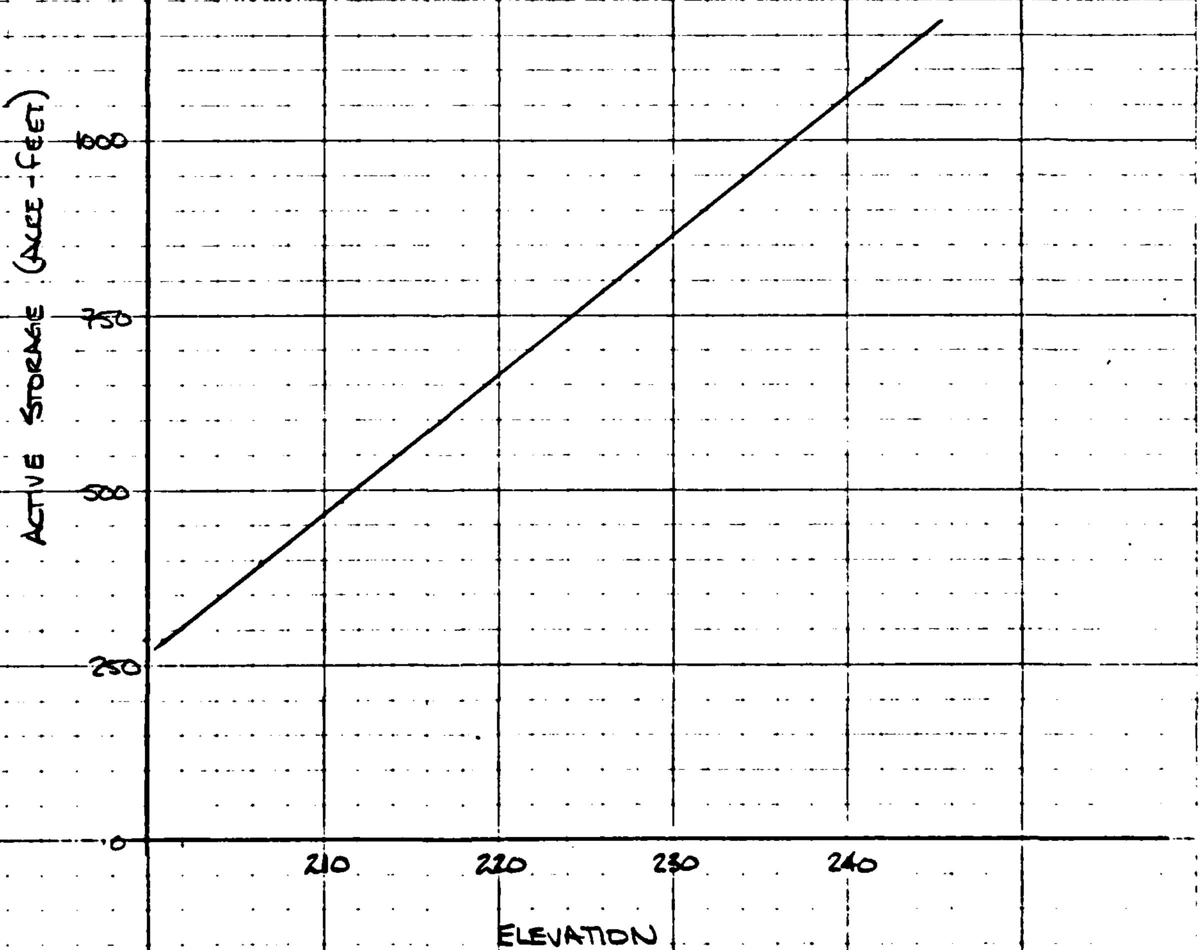
BASE FLOW

Based on Fall Creek @ Ithaca 04234000, average Aug. 1980 flow from 126 SQ MI is 42.2 cfs
∴ at Six Mile Brook Reservoir, base flow is

$$42.2 \text{ cfs} \left(\frac{47 \text{ SQ MI}}{126 \text{ SQ MI}} \right) = 16 \text{ cfs}$$

Based on other dam inspection reports, RTIOR = 1.5 and QRCSN, is 15%

STORAGE: Based on the assumption that the surface area will not significantly increase with changes in pool elevation.



METCALF & EDDY, ENGINEERS
NONREPRODUCIBLE GRID FORM 145

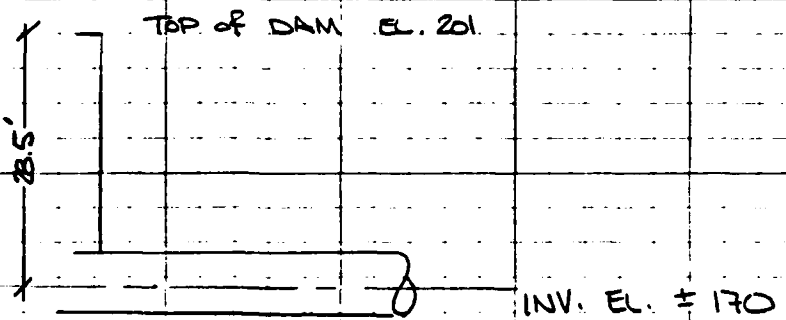
SPILLWAY The spillway is a concrete overflow weir. It is 65' long and discharge can be determined using a weir equation

$$Q = CLH^{3/2}$$

where $C = 3.1$ → See U.S. Dept. of Interior, Bureau of Rec. "Design of Small Dams"
$L = 65\text{ ft}$

H	Q (cfs)
0	0
1	202
2	570
3	1047
4	1612

LOW LEVEL OUTLET



$$H = 1.5\, V^2/2g$$

$$Q = VA$$

$$Q = \pi (2.5)^2 \left\{ \frac{64.4\,(28.5)}{1.5} \right\}^{1/2}$$

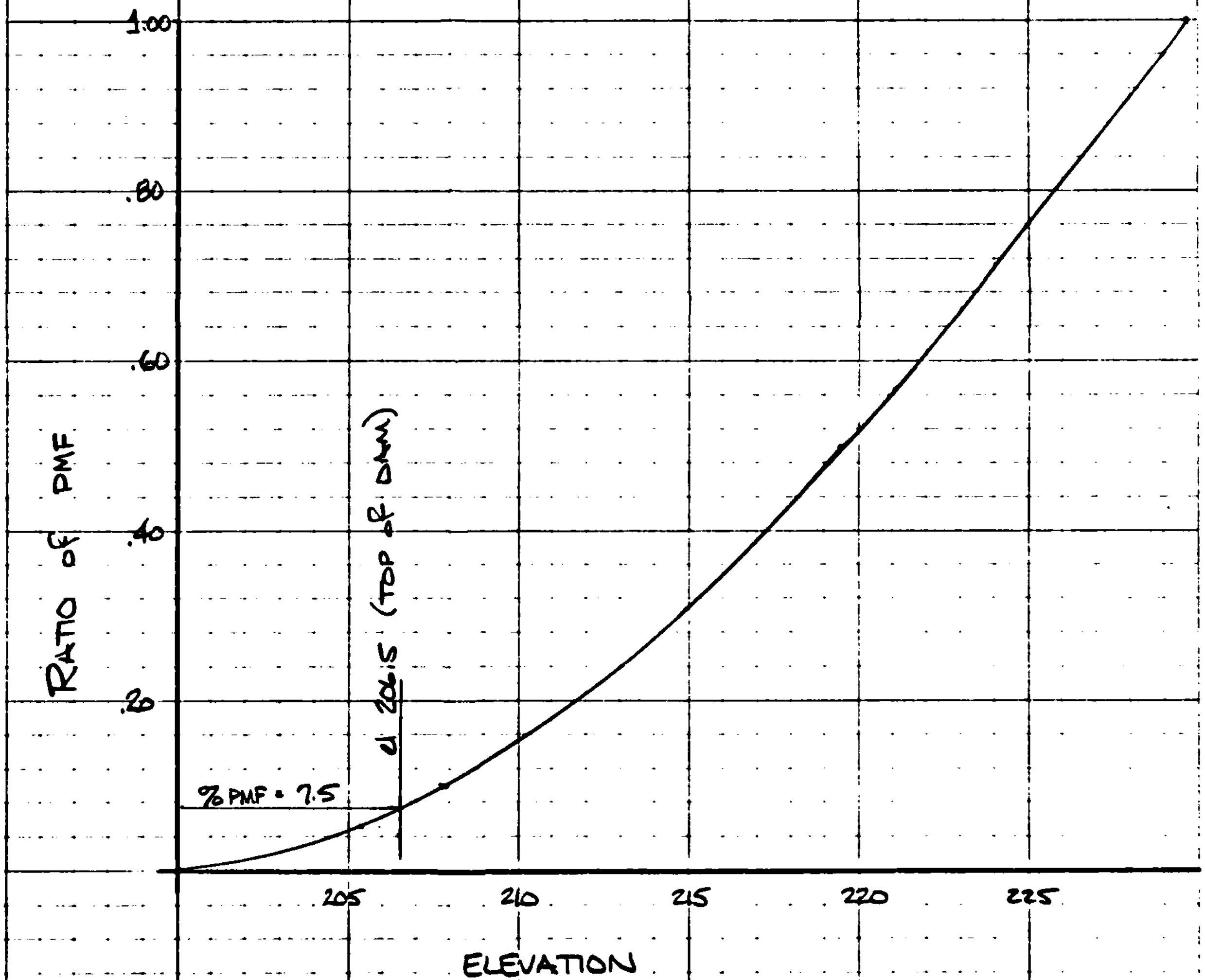
$$Q = 687 \text{ cfs}$$

NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

Project NY C of E PH I DAM INSP Acct. No. 7594 Page 4 of

Subject SIX MILE CREEK DAM Comptd. By M. NOWAK Date 8/26/81

Detail PERCENT PMF Ck'd. By Date



NONREPRODUCIBLE GRID FORM 145

METCALF & EDDY, ENGINEERS

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**********************************
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION      JULY 1978
  LAST MODIFICATION  25 SEP 78
**********************************
      1            A                NEW YORK C. OF E. PHASE 1 DAM INSPECTION
      2            A                     SIX MILE BROOK RES. - 30 FOOT DAM
      3            A                     FULL AND HALF PMF FLOOD ANALYSIS
      4            B     100       1       0       0       0       0       0       0       0       0
      5            B1      5
      6            J       1       4       1
      7            J1    .05      .1      .5     1.0
      8            K       0       1                       0       0       1       0
      9            K1                              INFLOW HYDROGRAPH
     10            M       1       1    47.0               0       0                       1
     11            P       0      21      93     106     117     124
     12            T                                                     1.0     0.1             .002
     13            W    8.03     0.6
     14            X      16    -.15     1.5
     15            K       1       1                       0       0       1       0
     16            K1                         ROUTED HYDROGRAPH AT DAM NO BREACH
     17            Y                               1       1
     18            Y1     10                                                 287       0
     19            SS    287     487     687     887    1087
     20            SE    201     211     221     231     241
     21            SS    201      65     3.1     1.5
     22            SD  206.5     2.7     1.5      21
     23            K      99
```

PREVIEW OF SEQUENCE OF STREAM NETWORK CALCULATIONS

RUNOFF HYDROGRAPH AT	1
ROUTE HYDROGRAPH TO	1
END OF NETWORK	

```
**********************************
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION      JULY 1978
  LAST MODIFICATION  25 SEP 78
**********************************

RUN  DATE:25 AUG 1981

                        NEW YORK C. OF E. PHASE 1 DAM INSPECTION
                          SIX MILE BROOK RES. - 30 FOOT DAM
                          FULL AND HALF PMF FLOOD ANALYSIS

                                  JOB SPECIFICATION
         NQ       NHR      NMIN     IDAY      IHR      IMIN     METRC     IPLT     IPRT    NSTAN
        100         1         0        0        0         0        0        0        0        0
                                   JOPER      NWT     LROPT    TRACE
                                       5        0         0        0

                          MULTI-PLAN ANALYSES TO BE PERFORMED
                               NPLAN= 1 NRTIO= 4 LRTIO= 1
          RTIOS=    0.05    0.10    0.50    1.00

          **********     **********     **********     **********     **********

                             SUB-AREA RUNOFF COMPUTATION

                                  INFLOW HYDROGRAPH

               ISTAQ    ICOMP    IECON    ITAPE     JPLT     JPRT    INAME   ISTAGE    IAUTO
                   1        0        0        0        0        0        1        0        0

                                   HYDROGRAPH DATA
         IHYDG    IUHG   TAREA    SNAP    TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
             1       1   47.00     0.0    47.00     0.0     0.0       0       1       0

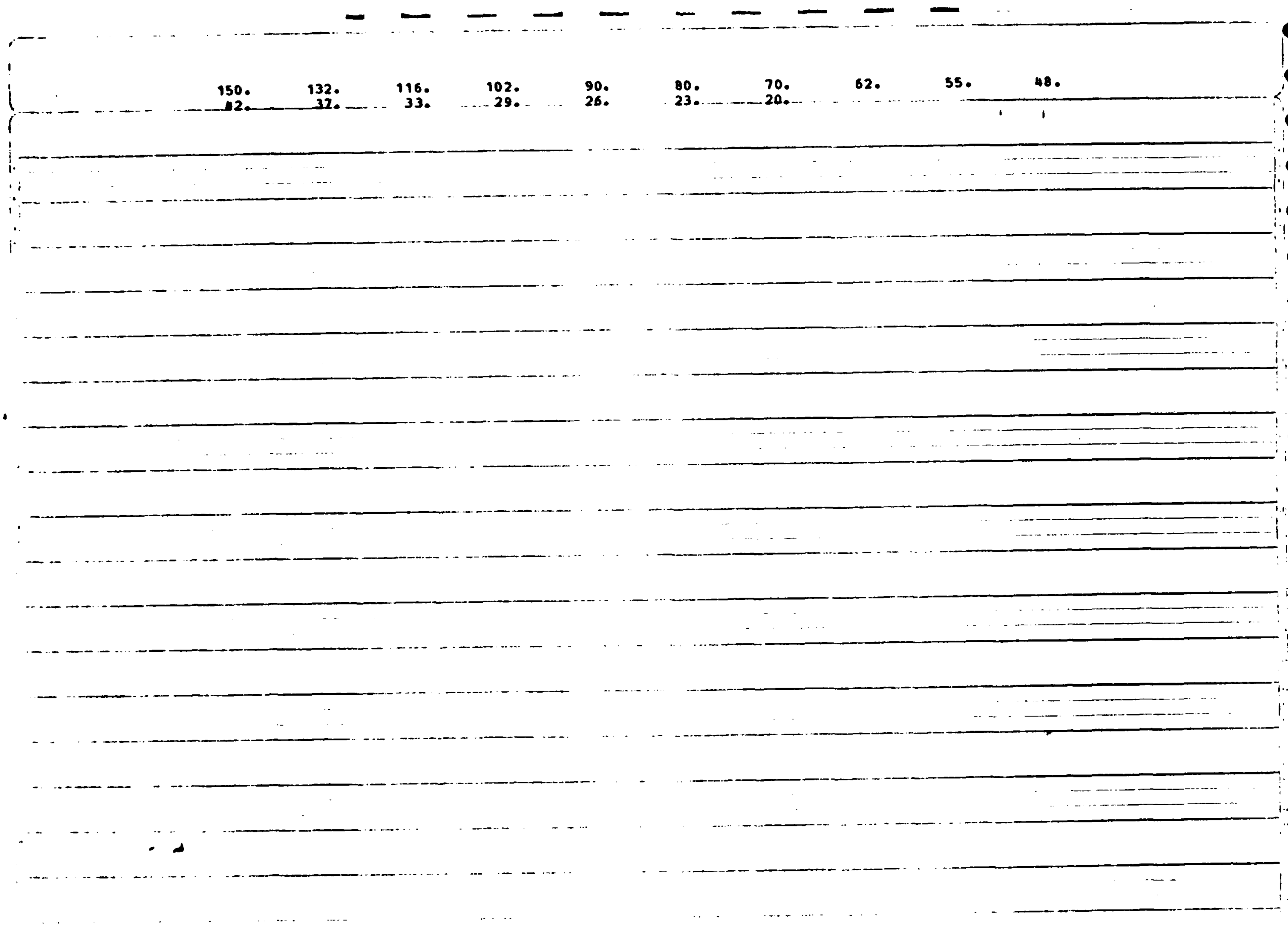
                                     PRECIP DATA
                  SPFE     PMS      R6      R12      R24      R48      R72      R96
                   0.0   21.00   93.00   106.00   117.00   124.00     0.0      0.0
TRSPC COMPUTED BY THE PROGRAM IS 0.848

                                      LOSS DATA
      LROPT   STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
          0     0.0     0.0    1.00     0.0     0.0    1.00    1.00    0.10     0.0    0.00

                                 UNIT HYDROGRAPH DATA
                            TP=  8.03      CP=0.60      NTA=  0

                                    RECESSION DATA
                     STRTQ=    16.00    QRCSN=    -0.15     RTIOR= 1.50
APPROXIMATE CLARK COEFFICIENTS FROM GIVEN SNYDER CP AND TP ARE TC= 8.94 AND R= 7.95 INTERVALS

          UNIT HYDROGRAPH 47 END-OF-PERIOD ORDINATES, LAG=   8.06 HOURS, CP= 0.60  VOL= 1.00
        95.      352.      709.     1116.     1542.     1916.     2175.     2310.     2293.     2108.
      1859.     1639.     1445.     1274.     1123.      990.      873.      769.      678.      598.
       527.      465.      410.      361.      318.      281.      247.      218.      192.      170.
```


150. 132. 116. 102. 90. 80. 70. 62. 55. 48.
42. 37. 33. 29. 26. 23. 20.

0

END-OF-PERIOD FLOW

MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q	MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q
1.01	1.00	1	0.01	0.00	0.01	15.	1.03	3.00	51	0.0	0.0	0.0	27927.
1.01	2.00	2	0.01	0.00	0.01	15.	1.03	4.00	52	0.0	0.0	0.0	24845.
1.01	3.00	3	0.01	0.00	0.01	14.	1.03	5.00	53	0.0	0.0	0.0	22074.
1.01	4.00	4	0.01	0.00	0.01	14.	1.03	6.00	54	0.0	0.0	0.0	19578.
1.01	5.00	5	0.01	0.00	0.01	13.	1.03	7.00	55	0.0	0.0	0.0	17331.
1.01	6.00	6	0.01	0.00	0.01	13.	1.03	8.00	56	0.0	0.0	0.0	15312.
1.01	7.00	7	0.02	0.00	0.02	12.	1.03	9.00	57	0.0	0.0	0.0	13508.
1.01	8.00	8	0.02	0.00	0.02	12.	1.03	10.00	58	0.0	0.0	0.0	11908.
1.01	9.00	9	0.02	0.00	0.02	11.	1.03	11.00	59	0.0	0.0	0.0	10499.
1.01	10.00	10	0.02	0.00	0.02	11.	1.03	12.00	60	0.0	0.0	0.0	9256.
1.01	11.00	11	0.02	0.00	0.02	11.	1.03	13.00	61	0.0	0.0	0.0	8160.
1.01	12.00	12	0.02	0.00	0.02	10.	1.03	14.00	62	0.0	0.0	0.0	7194.
1.01	13.00	13	0.10	0.00	0.10	10.	1.03	15.00	63	0.0	0.0	0.0	6342.
1.01	14.00	14	0.12	0.00	0.12	10.	1.03	16.00	64	0.0	0.0	0.0	5599.
1.01	15.00	15	0.15	0.00	0.15	10.	1.03	17.00	65	0.0	0.0	0.0	5376.
1.01	16.00	16	0.38	0.00	0.38	10.	1.03	18.00	66	0.0	0.0	0.0	5163.
1.01	17.00	17	0.14	0.02	0.12	12.	1.03	19.00	67	0.0	0.0	0.0	4958.
1.01	18.00	18	0.11	0.01	0.10	18.	1.03	20.00	68	0.0	0.0	0.0	4761.
1.01	19.00	19	0.01	0.00	0.01	27.	1.03	21.00	69	0.0	0.0	0.0	4571.
1.01	20.00	20	0.01	0.00	0.01	38.	1.03	22.00	70	0.0	0.0	0.0	4390.
1.01	21.00	21	0.01	0.00	0.01	50.	1.03	23.00	71	0.0	0.0	0.0	4215.
1.01	22.00	22	0.01	0.00	0.01	61.	1.04	0.0	72	0.0	0.0	0.0	4048.
1.01	23.00	23	0.01	0.00	0.01	69.	1.04	1.00	73	0.0	0.0	0.0	3887.
1.02	0.0	24	0.01	0.00	0.01	73.	1.04	2.00	74	0.0	0.0	0.0	3733.
1.02	1.00	25	0.13	0.03	0.10	77.	1.04	3.00	75	0.0	0.0	0.0	3584.
1.02	2.00	26	0.13	0.03	0.10	83.	1.04	4.00	76	0.0	0.0	0.0	3442.
1.02	3.00	27	0.13	0.03	0.10	98.	1.04	5.00	77	0.0	0.0	0.0	3305.
1.02	4.00	28	0.13	0.03	0.10	126.	1.04	6.00	78	0.0	0.0	0.0	3174.
1.02	5.00	29	0.13	0.03	0.10	167.	1.04	7.00	79	0.0	0.0	0.0	3048.
1.02	6.00	30	0.13	0.03	0.10	221.	1.04	8.00	80	0.0	0.0	0.0	2927.
1.02	7.00	31	0.39	0.29	0.10	307.	1.04	9.00	81	0.0	0.0	0.0	2810.
1.02	8.00	32	0.39	0.29	0.10	464.	1.04	10.00	82	0.0	0.0	0.0	2699.
1.02	9.00	33	0.39	0.29	0.10	711.	1.04	11.00	83	0.0	0.0	0.0	2591.
1.02	10.00	34	0.39	0.29	0.10	1058.	1.04	12.00	84	0.0	0.0	0.0	2488.
1.02	11.00	35	0.39	0.29	0.10	1505.	1.04	13.00	85	0.0	0.0	0.0	2390.
1.02	12.00	36	0.39	0.29	0.10	2042.	1.04	14.00	86	0.0	0.0	0.0	2295.
1.02	13.00	37	1.66	1.56	0.10	2760.	1.04	15.00	87	0.0	0.0	0.0	2203.
1.02	14.00	38	1.99	1.89	0.10	3865.	1.04	16.00	88	0.0	0.0	0.0	2116.
1.02	15.00	39	2.48	2.38	0.10	5547.	1.04	17.00	89	0.0	0.0	0.0	2032.
1.02	16.00	40	6.29	6.19	0.10	8303.	1.04	18.00	90	0.0	0.0	0.0	1951.
1.02	17.00	41	2.32	2.22	0.10	12447.	1.04	19.00	91	0.0	0.0	0.0	1874.
1.02	18.00	42	1.82	1.72	0.10	17639.	1.04	20.00	92	0.0	0.0	0.0	1799.
1.02	19.00	43	0.20	0.10	0.10	23294.	1.04	21.00	93	0.0	0.0	0.0	1728.
1.02	20.00	44	0.20	0.10	0.10	28758.	1.04	22.00	94	0.0	0.0	0.0	1659.
1.02	21.00	45	0.20	0.10	0.10	33280.	1.04	23.00	95	0.0	0.0	0.0	1593.
1.02	22.00	46	0.20	0.10	0.10	36221.	1.05	0.0	96	0.0	0.0	0.0	1530.
1.02	23.00	47	0.20	0.10	0.10	37348.	1.05	1.00	97	0.0	0.0	0.0	1469.
1.03	0.0	48	0.20	0.10	0.10	36658.	1.05	2.00	98	0.0	0.0	0.0	1411.
1.03	1.00	49	0.0	0.0	0.0	34366.	1.05	3.00	99	0.0	0.0	0.0	1355.
1.03	2.00	50	0.0	0.0	0.0	31209.	1.05	4.00	100	0.0	0.0	0.0	1301.

SUM 22.08 18.47 3.61 622502.
(561.)(469.)(92.)(17627.32)

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	37348.	34643.	20544.	8625.	621838.
CMS	1058.	981.	582.	244.	17609.

INCHES	6.86	16.26	20.48	20.51
MM	174.16	413.12	520.31	521.02
AC-FT	17178.	40749.	51321.	51392.

THOUS CU M		21189.	50263.	63304.	63391.

HYDROGRAPH AT STA 1 FOR PLAN 1, RTIO 1

1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	0.	0.	0.	1.	1.	1.	2.
2.	3.	3.	4.	4.	4.	5.	6.	8.	11.
15.	23.	36.	53.	75.	102.	138.	193.	277.	415.
622.	882.	1165.	1438.	1664.	1811.	1867.	1833.	1718.	1560.
1396.	1242.	1104.	979.	867.	766.	675.	595.	525.	463.
408.	360.	317.	280.	269.	258.	248.	238.	229.	219.
211.	202.	194.	187.	179.	172.	165.	159.	152.	146.
141.	135.	130.	124.	119.	115.	110.	106.	102.	98.
94.	90.	86.	83.	80.	76.	73.	71.	68.	65.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1867.	1732.	1027.	431.	31092.
CMS	53.	49.	29.	12.	880.
INCHES		0.34	0.81	1.02	1.03
MM		8.71	20.66	26.02	26.05
AC-FT		859.	2037.	2566.	2570.
THOUS CU M		1059.	2513.	3165.	3170.

HYDROGRAPH AT STA 1 FOR PLAN 1, RTIO 2

2.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	2.	3.	4.
5.	6.	7.	7.	8.	8.	10.	13.	17.	22.
31.	46.	71.	106.	151.	204.	276.	387.	555.	830.
1245.	1764.	2329.	2876.	3328.	3622.	3735.	3666.	3437.	3121.
2793.	2485.	2207.	1958.	1733.	1531.	1351.	1191.	1050.	926.
816.	719.	634.	560.	538.	516.	496.	476.	457.	439.
422.	405.	389.	373.	358.	344.	331.	317.	305.	293.
281.	270.	259.	249.	239.	229.	220.	212.	203.	195.
187.	180.	173.	166.	159.	153.	147.	141.	135.	130.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	3735.	3464.	2054.	862.	62184.
CMS	106.	98.	58.	24.	17[illegible]1.
INCHES		0.69	1.63	2.05	2.[illegible]5
MM		17.42	41.31	52.03	52.10
AC-FT		1718.	4075.	5132.	51[illegible]9.
THOUS CU M		2119.	5026.	6330.	6339.

HYDROGRAPH AT STA 1 FOR PLAN 1, RTIO 3

8.	7.	7.	7.	7.	6.	6.	6.	6.	5.
5.	5.	5.	5.	5.	5.	6.	9.	13.	19.
25.	30.	34.	37.	38.	42.	49.	63.	83.	110.
153.	232.	356.	529.	753.	1021.	1380.	1933.	2774.	4151.
6223.	8819.	11647.	14379.	16640.	18110.	18674.	18329.	17183.	15605.
13963.	12423.	11037.	9789.	8666.	7656.	6754.	5954.	5249.	4628.
4080.	3597.	3171.	2799.	2688.	2581.	2479.	2380.	2286.	2195.
2108.	2024.	1944.	1866.	1792.	1721.	1653.	1587.	1524.	1463.
1405.	1349.	1296.	1244.	1195.	1147.	1102.	1058.	1016.	976.
937.	900.	864.	829.	797.	765.	734.	705.	677.	650.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	18674.	17321.	10272.	4312.	310919.
CMS	529.	490.	291.	122.	8804.

```
                 INCHES              3.43     8.13    10.24        10.26
                     MM             87.08   206.56   260.15       260.51
                  AC-FT             8589.   20374.   25661.       25696.
             THOUS CU M            10595.   25131.   31652.       31695.

                        HYDROGRAPH AT STA      1 FOR PLAN 1, RTIO 4
      15.      15.      14.      14.      13.      13.      12.      12.      11.      11.
      11.      10.      10.      10.      10.      10.      12.      18.      27.      38.
      50.      61.      69.      73.      77.      83.      98.     126.     167.     221.
     307.     464.     711.    1058.    1505.    2042.    2760.    3865.    5547.    8303.
   12447.   17639.   23294.   28758.   33280.   36221.   37348.   36658.   34366.   31209.
   27927.   24845.   22074.   19578.   17331.   15312.   13508.   11908.   10499.    9256.
    8160.    7194.    6342.    5599.    5376.    5163.    4958.    4761.    4571.    4390.
    4215.    4048.    3887.    3733.    3584.    3442.    3305.    3174.    3048.    2927.
    2810.    2699.    2591.    2488.    2390.    2295.    2203.    2116.    2032.    1951.
    1874.    1799.    1728.    1659.    1593.    1530.    1469.    1411.    1355.    1301.

                           PEAK   6-HOUR   24-HOUR   72-HOUR   TOTAL VOLUME
                    CFS  37348.   34643.    20544.     8625.        621838.
                    CMS   1058.     981.      582.      244.         17609.
                 INCHES             6.86     16.26     20.48          20.51
                     MM           174.16    413.12    520.31         521.02
                  AC-FT           17178.    40749.    51321.         51392.
             THOUS CU M           21189.    50263.    63304.         63391.


        **********          **********          **********          **********          **********

                                 HYDROGRAPH ROUTING

                        ROUTED HYDROGRAPH AT DAM NO BREACH

                        ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME  ISTAGE   IAUTO
                            1       1       0       0       0       0       1       0       0
                                              ROUTING DATA
                 QLOSS  CLOSS     AVG    IRES   ISAME    IOPT    IPMP            LSTR
                   0.0    0.0     0.0       1       1       0       0               0

                        NSTPS   NSTDL     LAG   AMSKK       X     TSK   STORA  ISPRAT
                           10       0       0     0.0     0.0     0.0    287.       0

        CAPACITY=      287.      487.      687.      887.     1087.

       ELEVATION=      201.      211.      221.      231.      241.

                     CREL    SPWID    COQW    EXPW   ELEVL    COQL   CAREA    EXPL
                    201.0     65.0     3.1     1.5     0.0     0.0     0.0     0.0
                                                 DAM DATA
                                     TOPEL    COQD    EXPD   DAMWID
                                     206.5     2.7     1.5      21.

                                 STATION      1, PLAN 1, RATIO 1

                               END-OF-PERIOD HYDROGRAPH ORDINATES
```

OUTFLOW

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

0.	0.	0.	0.	0.	0.	0.	1.	1.	1.
1.	1.	2.	2.	3.	3.	3.	4.	5.	6.
9.	13.	20.	31.	49.	72.	103.	146.	213.	321.
495.	739.	1026.	1317.	1573.	1757.	1850.	1850.	1766.	1624.
1463.	1307.	1164.	1036.	920.	815.	722.	638.	564.	499.
441.	390.	345.	306.	280.	267.	256.	246.	236.	227.
218.	209.	201.	193.	185.	178.	171.	164.	158.	152.
146.	140.	135.	129.	124.	119.	115.	110.	106.	102.
98.	94.	90.	87.	83.	80.	77.	74.	71.	68.

STORAGE

287.	287.	287.	287.	287.	287.	287.	287.	287.	287.
287.	287.	287.	287.	287.	287.	287.	287.	287.	287.
288.	288.	288.	288.	288.	288.	288.	288.	289.	289.
289.	290.	291.	293.	295.	297.	300.	303.	308.	314.
323.	335.	346.	357.	366.	372.	375.	375.	372.	367.
362.	357.	351.	347.	342.	338.	334.	330.	327.	324.
321.	318.	316.	313.	312.	311.	310.	310.	309.	309.
308.	307.	307.	306.	306.	305.	305.	304.	304.	304.
303.	303.	302.	302.	301.	301.	301.	300.	300.	300.
299.	299.	299.	298.	298.	298.	298.	297.	297.	297.

STAGE

201.0	201.0	201.0	201.0	201.0	201.0	201.0	201.0	201.0	201.0
201.0	201.0	201.0	201.0	201.0	201.0	201.0	201.0	201.0	201.0
201.0	201.0	201.0	201.0	201.1	201.1	201.1	201.1	201.1	201.1
201.1	201.2	201.2	201.3	201.4	201.5	201.6	201.8	202.0	202.4
202.8	203.4	204.0	204.5	204.9	205.2	205.4	205.4	205.3	205.0
204.8	204.5	204.2	204.0	203.8	203.5	203.3	203.2	203.0	202.8
202.7	202.6	202.4	202.3	202.2	202.2	202.2	202.1	202.1	202.1
202.1	202.0	202.0	202.0	201.9	201.9	201.9	201.9	201.9	201.8
201.8	201.8	201.8	201.7	201.7	201.7	201.7	201.7	201.7	201.6
201.6	201.6	201.6	201.6	201.6	201.5	201.5	201.5	201.5	201.5

PEAK OUTFLOW IS 1850. AT TIME 48.00 HOURS

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1850.	1727.	1026.	430.	30976.
CMS	52.	49.	29.	12.	877.
INCHES		0.34	0.81	1.02	1.02
MM		8.68	20.62	25.93	25.95
AC-FT		857.	2034.	2558.	2560.
THOUS CU M		1057.	2509.	3155.	3158.

STATION 1, PLAN 1, RATIO 2

END-OF-PERIOD HYDROGRAPH ORDINATES

OUTFLOW

0.	0.	0.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	1.	1.	2.
2.	3.	4.	5.	6.	6.	7.	9.	11.	14.
19.	28.	44.	70.	107.	156.	220.	310.	450.	677.
1039.	1538.	2113.	2689.	3202.	3550.	3716.	3696.	3509.	3215.
2892.	2583.	2304.	2047.	1817.	1610.	1424.	1259.	1112.	983.
869.	768.	679.	601.	552.	529.	508.	488.	469.	450.
432.	415.	399.	383.	368.	354.	340.	326.	314.	301.

289.	278.	267.	257.	246.	237.	227.	218.	210.	202.
198.	186.	179.	172.	165.	159.	152.	146.	141.	135.

STORAGE

287.	287.	287.	287.	287.	287.	288.	288.	288.	288.
288.	288.	288.	288.	288.	288.	288.	288.	288.	288.
288.	288.	288.	289.	289.	289.	289.	289.	290.	290.
291.	292.	294.	297.	300.	304.	308.	314.	321.	332.
347.	365.	383.	399.	412.	421.	424.	424.	420.	413.
405.	397.	388.	381.	374.	367.	361.	355.	349.	345.
340.	336.	332.	328.	326.	325.	324.	323.	322.	321.
320.	319.	319.	318.	317.	316.	315.	315.	314.	313.
312.	312.	311.	310.	310.	309.	309.	308.	308.	307.
306.	306.	305.	305.	305.	304.	304.	303.	303.	302.

STAGE

201.0	201.0	201.0	201.0	201.0	201.0	201.0	201.0	201.0	201.0
201.0	201.0	201.0	201.0	201.0	201.0	201.0	201.0	201.0	201.0
201.1	201.1	201.1	201.1	201.1	201.1	201.1	201.1	201.1	201.2
201.2	201.3	201.4	201.5	201.7	201.8	202.1	202.3	202.7	203.2
204.0	204.9	205.8	206.6	207.3	207.7	207.9	207.8	207.6	207.3
206.9	206.5	206.1	205.7	205.3	205.0	204.7	204.4	204.1	203.9
203.6	203.4	203.2	203.1	203.0	202.9	202.9	202.8	202.8	202.7
202.7	202.6	202.6	202.5	202.5	202.5	202.4	202.4	202.3	202.3
202.3	202.2	202.2	202.2	202.1	202.1	202.1	202.1	202.0	202.0
202.0	201.9	201.9	201.9	201.9	201.9	201.8	201.8	201.8	201.8

PEAK OUTFLOW IS 3716. AT TIME 47.00 HOURS

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	3716.	3456.	2052.	860.	62000.
CMS	105.	98.	58.	24.	1756.
INCHES		0.68	1.62	2.04	2.05
MM		17.37	41.27	51.90	51.95
AC-FT		1713.	4070.	5119.	5124.
THOUS CU M		2114.	5021.	6315.	6320.

STATION 1, PLAN 1, RATIO 3

END-OF-PERIOD HYDROGRAPH ORDINATES

OUTFLOW

1.	2.	4.	5.	5.	6.	6.	6.	6.	6.
6.	6.	5.	5.	5.	5.	5.	6.	7.	11.
15.	21.	26.	31.	34.	37.	41.	49.	63.	84.
116.	171.	267.	416.	623.	882.	1213.	1697.	2452.	3712.
5685.	8261.	11123.	13932.	16315.	17933.	18638.	18429.	17398.	15869.
14231.	12679.	11274.	10013.	8875.	7853.	6938.	6124.	5406.	4773.
4215.	3722.	3288.	2908.	2706.	2620.	2513.	2415.	2319.	2227.
2139.	2055.	1973.	1895.	1820.	1748.	1679.	1613.	1549.	1488.
1429.	1373.	1319.	1266.	1216.	1168.	1122.	1078.	1035.	994.
955.	917.	881.	846.	813.	781.	750.	721.	692.	665.

STORAGE

288.	288.	288.	289.	289.	289.	289.	289.	289.	289.
289.	289.	289.	289.	289.	289.	289.	289.	289.	290.
291.	291.	292.	293.	293.	293.	294.	295.	296.	298.
301.	305.	311.	319.	329.	341.	353.	370.	393.	424.
465.	510.	555.	595.	627.	647.	656.	654.	641.	621.
[illegible]	577.	557.	538.	520.	503.	487.	473.	459.	447.

435.	424.	414.	405.	400.	398.	395.	392.	389.	386.
384.	381.	379.	376.	374.	371.	369.	367.	365.	363.
361.	359.	357.	355.	353.	352.	350.	348.	347.	345.

343.	342.	340.	339.	338.	336.	335.	334.	333.	331.

STAGE

201.0	201.1	201.1	201.1	201.1	201.1	201.1	201.1	201.1	201.1
201.1	201.1	201.1	201.1	201.1	201.1	201.1	201.1	201.1	201.1
201.2	201.2	201.3	201.3	201.3	201.3	201.3	201.4	201.5	201.6
201.7	201.9	202.2	202.6	203.1	203.7	204.3	205.1	206.3	207.9
209.9	212.1	214.4	216.4	218.0	219.0	219.5	219.3	218.7	217.7
216.6	215.5	214.5	213.5	212.6	211.8	211.0	210.3	209.6	209.0
208.4	207.9	207.4	206.9	206.6	206.5	206.4	206.2	206.1	206.0
205.8	205.7	205.6	205.5	205.3	205.2	205.1	205.0	204.9	204.8
204.7	204.6	204.5	204.4	204.3	204.2	204.1	204.1	204.0	203.9
203.8	203.7	203.7	203.6	203.5	203.5	203.4	203.3	203.3	203.2

PEAK OUTFLOW IS 18638. AT TIME 47.00 HOURS

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	18638.	17269.	10262.	4306.	310390.
CMS	528.	489.	291.	122.	8789.
INCHES		3.42	8.12	10.23	10.24
MM		86.81	206.35	259.78	260.07
AC-FT		8563.	20354.	25624.	25652.
THOUS CU M		10562.	25106.	31607.	31641.

STATION 1, PLAN 1, RATIO 4

END-OF-PERIOD HYDROGRAPH ORDINATES

OUTFLOW

3.	6.	9.	11.	12.	12.	12.	12.	12.	12.
11.	11.	11.	10.	10.	10.	10.	12.	16.	23.
33.	45.	56.	65.	71.	77.	85.	103.	133.	177.
244.	362.	565.	876.	1299.	1823.	2492.	3511.	5091.	7637.
11605.	16768.	22472.	28059.	32773.	35948.	37297.	36823.	34712.	31628.
28350.	25249.	22447.	19932.	17661.	15622.	13795.	12174.	10743.	9481.
8368.	7385.	6519.	5760.	5387.	5227.	5001.	4810.	4617.	4435.
4259.	4091.	3929.	3774.	3624.	3481.	3344.	3211.	3085.	2963.
2846.	2734.	2626.	2524.	2424.	2328.	2236.	2147.	2062.	1981.
1903.	1827.	1755.	1686.	1619.	1555.	1494.	1435.	1378.	1324.

STORAGE

288.	289.	290.	290.	290.	290.	290.	290.	290.	290.
290.	290.	290.	290.	290.	290.	290.	290.	291.	292.
293.	294.	296.	296.	297.	297.	298.	300.	302.	305.
310.	317.	327.	340.	356.	374.	394.	420.	453.	500.
562.	633.	703.	765.	815.	847.	861.	856.	835.	803.
769.	734.	702.	672.	644.	618.	593.	570.	549.	530.
512.	495.	480.	466.	459.	456.	451.	448.	444.	440.
436.	433.	429.	426.	422.	419.	416.	413.	410.	407.
404.	401.	398.	395.	392.	389.	386.	384.	381.	379.
376.	374.	372.	369.	367.	365.	363.	361.	359.	357.

STAGE

201.1	201.1	201.1	201.1	201.2	201.2	201.2	201.2	201.2	201.1
201.1	201.1	201.1	201.1	201.1	201.1	201.1	201.2	201.2	201.2
201.3	201.4	201.4	201.5	201.5	201.5	201.6	201.6	201.8	201.9
202.1	202.5	203.0	203.7	204.5	205.3	206.3	207.6	209.3	211.6

214.7	218.3	221.8	224.9	227.4	229.0	229.7	229.5	228.4	226.8
225.1	223.4	221.8	220.3	218.9	217.5	216.3	215.2	214.1	213.1
212.2	211.4	210.6	210.0	209.6	209.4	209.2	209.0	208.8	208.6

208.5	208.3	208.1	207.9	207.8	207.6	207.4	207.3	207.1	207.0
206.8	206.7	206.5	206.4	206.2	206.1	206.0	205.8	205.7	205.6
205.5	205.3	205.2	205.1	205.0	204.9	204.8	204.7	204.6	204.5

PEAK OUTFLOW IS 37297. AT TIME 47.00 HOURS

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	37297.	34566.	20531.	8615.	621003.
CMS	1056.	979.	581.	244.	17585.
INCHES		6.84	16.25	20.46	20.49
MM		173.77	412.85	519.73	520.32
AC-FT		17140.	40722.	51264.	51323.
THOUS CU M		21142.	50230.	63233.	63306.

********** ********** ********** ********** **********

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

OPERATION	STATION	AREA	PLAN	RATIO 1 0.05	RATIO 2 0.10	RATIO 3 0.50	RATIO 4 1.00
				RATIOS APPLIED TO FLOWS			
HYDROGRAPH AT	1	47.00	1	1867.	3735.	18674.	37348.
		(121.73)		(52.88)(	105.76)(	528.79)(	1057.58)(
ROUTED TO	1	47.00	1	1850.	3716.	18638.	37297.
		(121.73)		(52.39)(	105.22)(	527.76)(	1056.13)(

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	201.00	201.00	206.50
STORAGE	287.	287.	397.
OUTFLOW	0.	0.	2599.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
0.05	205.38	0.0	375.	1850.	0.0	48.00	0.0
0.10	207.87	1.37	424.	3716.	8.00	47.00	0.0
0.50	219.47	12.97	656.	18638.	27.00	47.00	0.0
1.00	229.69	23.19	861.	37297.	46.00	47.00	0.0

APPENDIX D

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APPENDIX E

PREVIOUS INSPECTION REPORTS AND AVAILABLE DOCUMENTS

S 73 - 71 o. Oswego

Acc 382

(NOTICE: After filling out one of these forms as completely as possible for each dam in your district, return it at once to the Conservation Commission, Albany.)

STATE OF NEW YORK
CONSERVATION COMMISSION
ALBANY

DAM REPORT

Aug 16, 1912
(Date)

CONSERVATION COMMISSION,

DIVISION OF WATERS.

GENTLEMEN:

I have the honor to make the following report in relation to the structure known as the Thirty foot Dam.

This dam is situated upon the Sixmile Creek (Give name of stream)

in the Town of Ithaca, Tompkins County,

about 1 mile (State distance) from the Village or City of Ithaca

The distance down (Up or down) stream from the dam, to the Van Ulca (?) Dam (Give name of nearest important stream or of a bridge);

is about 1 mile (State distance)

The dam is now owned by City of Ithaca (Give name and address in full)

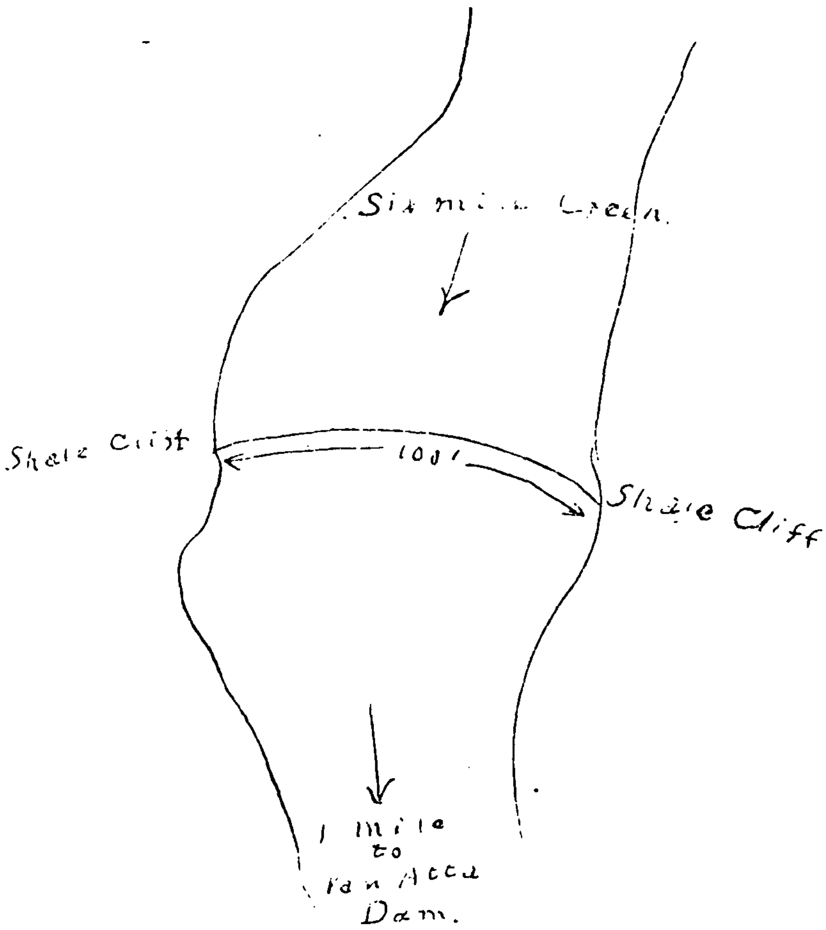
and was built in or about the year 1909, and was extensively repaired or reconstructed during the year.........

As it now stands, the spillway portion of this dam is built of Concrete (State whether of masonry, concrete or timber)

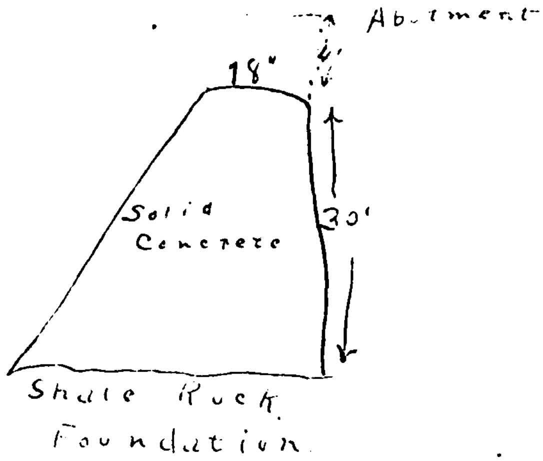
and the other portions are built of Concrete (State whether of masonry, concrete, earth or timber with or without rock fill)

As nearly as I can learn, the character of the foundation bed under the spillway portion of the dam is Shale and under the remaining portions such foundation bed is Shale.

(In the space below, make a third sketch showing the general plan of the dam, and its approximate position in relation to buildings or other conspicuous objects in the vicinity.)



[illegible] space below, make one sketch showing the form and dimensions of a cross section through the spillway [illegible] of the dam and cutting the abutment, and a second sketch showing the same information for a cross section through the other part of the dam. Show particularly the greatest height of the dam above the stream bed, its thickness at the top, and thickness at the bottom, as nearly as you can learn.



The total length of this dam is 172 feet. The spillway or waste weir portion, is about 150 feet long, and the crest of the spillway is about .... feet below the abutment.

The number, size and location of discharge pipes, waste pipes or gates which may be used for drawing off the water from behind the dam, are as follows: ....

....

At the time of this inspection the water level above the dam was .... ft. 1 in. below / above the crest of the spillway.

(State briefly, in the space below, whether, in your judgment, this dam is in good condition, or bad condition, describing particularly any leaks or cracks or erosions which you may have observed.)

Dam is in good condition. City keeps it in good repair.

Reported by Louis A. Callender
(Signature)

903 University Av
(Address—Street and number, P. O. Box or R. F. D. route)

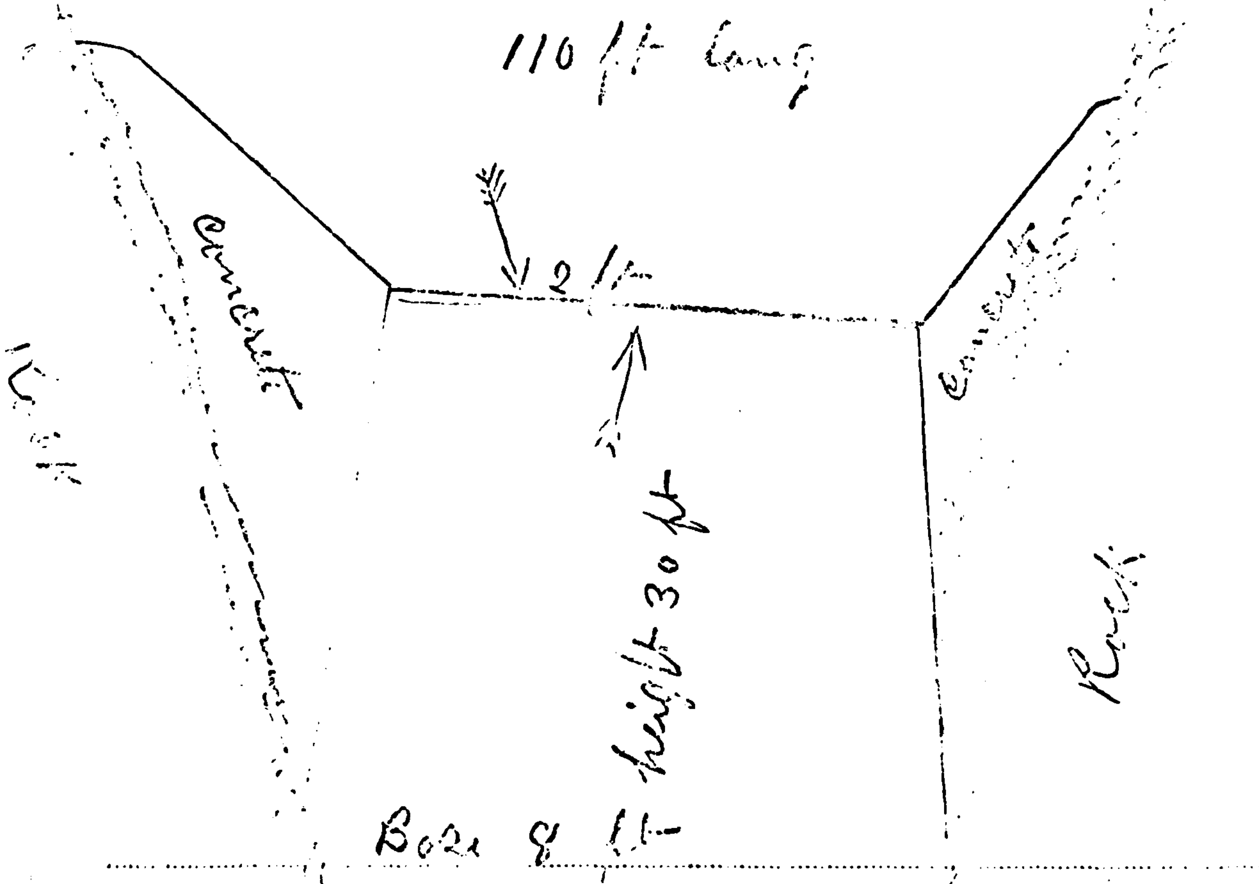
Syracuse N.Y.
(Name of place)

Six Mile Creek Dam.

Fill out a form as complete as possible for each dam in every district and send to State Conservation Commission, Albany, N. Y.

1. Name and address of owners City Water Works Co. Ithaca N.Y.
2. Date of construction 1902.
3. Uses of impounded water City Water Main.
4. Character of foundation bed Rock.
5. Material of waste spill Concrete.
6. Length of waste and depth below dam Waste 60ft. Depth below dam 5ft.
7. Total length of dam including waste 110ft.
8. Material of dam Concrete.
9. Discharges, size and location Two large Iron pipes to pumping station 1/2 mi.

Below sketch section of waste and section of dam, with greatest heights and top thickness and bottom thickness. On opposite side sketch general plan of dam and give distance from a bridge or from a tributary stream.



Nearest town Ithaca

(Signature, address and date.)

John B. [illegible]
Ithaca N.Y.

Rock Feb 12th 1913 Bridge [illegible]

37-1-4 (7/76)

NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

DAM INSPECTION REPORT

(By Visual Inspection)

City of Ithaca

Dam Number	River Basin	Town	County	Hazard Class*	Date & Inspector
710	Oswego	Ithaca	Tompkins	C	11/2[illegible]/80 [illegible]

Type of Construction

- ☐ Earth w/concrete spillway
- ☐ Earth w/drop inlet pipe
- ☐ Earth w/stone or riprap spillway
- ☒ Concrete w/drop inlet pipe (inoperable)
- ☐ Stone
- ☐ Timber

Use

- ☐ Water Supply
- ☐ Power
- ☐ Recreation
- ☐ Fish and Wildlife
- ☐ Farm Pond
- ☒ No Apparent Use-Abandoned

Originally to be used for water supply.

Estimated Impoundment Size

- ☐ 1-5 acres
- ☐ 5-10 acres
- ☒ Over 10 acres (10+ Acres)

Estimated Height of Dam above Streambed

- ☐ Under 10 feet
- ☐ 10-25 feet
- ☒ Over 25 feet 30'

Condition of Spillway

- ☒ Service satisfactory
- ☐ In need of repair or maintenance
- ☐ Auxiliary satisfactory
- ☐ In need of repair or maintenance
- ☒ None

Explain: 4"-6" of water over dam. Hard to see because of water and high terrain. Reinspect

Condition of Non-Overflow Section

- ☒ Satisfactory
- ☐ In need of repair or maintenance

Explain: None. Drop Inlet plugged

Condition of Mechanical Equipment

- ☐ Satisfactory
- ☐ In need of repair or maintenance

Explain:

Evaluation (From Visual Inspection)

- ☒ No defects observed beyond normal maintenance
- ☐ Repairs required beyond normal maintenance

*Explain Hazard Class, if Necessary

"C" Hazard [illegible] because homes would be flooded if dam failed

STATE OF NEW YORK

DEPARTMENT OF

State Engineer and Surveyor

ALBANY

Report of a Structure Impounding Water

To assist in carrying out the provisions of Section 22 of the Conservation Law, being Chapter LXV of the Consolidated Laws of New York State, relating to safeguarding life and property and the erection, reconstruction, or maintenance of structures for impounding water, owners of such structures are requested to fill out as completely as possible this report form for each such dam or reservoir owned within the State of New York for which no plans or reports relative thereto are on file in this Department, and to return this report form, together with prints or photographs explanatory thereof to this department.

1. The structure is on Six Mile Creek flowing into Cayuga Inlet in the Town of Ithaca County of Tompkins and 3/4 Mile upstream from southerly City limits of Ithaca, N.Y. (Give exact distance and direction from a well-known bridge, dam, village main cross-roads or mouth of a stream)

2. Is any part of the structure built upon or does its pond flood any State lands? No

3. The name and address of the owner is City of Ithaca

4. The structure is used for Not In Use. Formerly City water supply

5. The material of the right bank, in the direction with the current, is Shale Rock; at the spillway crest elevation this material has a top slope of ...... inches vertical to a foot horizontal on the center line of the structure, a vertical thickness at this elevation of Unknown feet, and the top surface extends for a vertical height of 40 feet above the spillway crest.
at spillway, banks are vertical or overhanging for a height of 40's above crest

6. The material of the left bank is Shale Rock; ~~horizontal~~ vertical ~~inches to a foot horizontal~~, a thickness of Unknown feet and a height of 40 feet.

7. The natural material of the bed on which the structure rests is (clay, sand, gravel, boulders, granite, shale, slate, limestone, etc.) Shale Rock

8. State the character of the bed and the banks in respect to the hardness, perviousness, water bearing, effect of exposure to air and to water, uniformity, etc. Shale Rock, Fairly Hard, Nearly Impervious, Shale probably water bearing under dam, weather has little effect, material quite uniform. Term "Shale" includes thin alternate layers of fine grained sandstone or "flagstone".

9. If the bed is in layers, are the layers horizontal or inclined? horizontal If inclined what is the direction of the horizontal outcropping relative to the axis of the main structure and the inclination and direction of the layers in a plane perpendicular to the horizontal outcropping?

10. What is the thickness of the layers? Two To Twelve Inches

11. Are there any porous seams or fissures? yes Mostly Fine cracks

12. The watershed at the above structure and draining into the pond formed thereby is 43 square miles.

13. The pond area at the spillway crest elevation is 20 acres and the pond impounds 12,500,000 cubic feet of water. (practically silted up)

14. The maximum known flow of the stream at the structure was 8,500 cubic feet per second on June 21, 1905 (Date)

15. Has the spillway capacity ever been exceeded by a high flow? No

Can any possible flood flow from the pond otherwise than through the wastes noted under 17 and 18 of this report? No If so, give the location, the length and the elevation relative to the spillway crest and the character and slopes of the ground of such possible wastes

16. State if any damage to life or to any buildings, roads or other property could be caused by any possible failure of the above structure. Describe the location, the character and the use of buildings below the structure which might be damaged by any failure of the structure; of roads adjacent to or crossing the stream below the structure, giving the lowest elevation of the roadway above the stream bed and giving the shape, the height and the width of stream openings; and of any embankments or steep slopes that any flood could pass over. Also indicate the character and use made of the ground below the structure.

No. Amount of water stored too small and Channel is clear below.

17. Wastes. The spillway of the above structure is 63 feet long in the clear; the waters are held at the right end by a brick wall built into the shale the top of which is 5'-6" feet above the spillway crest, and has a top width of 10 feet; and at the left end by a ditto, the top of which is 5'-6" feet above the spillway crest, and has a top width of 10 feet.

18. There is also for flood discharge a ~~pipe~~ NONE – Flood discharges over spillway inches inside diameter and the bottom is feet below the spillway crest; and a (sluice, gate outlet) feet wide in the clear by feet high, and the bottom is feet below the spillway crest.

19. Apron. None ~~Below the~~ spillway there is an apron built of...

feet wide and........................feet thick. The downstream end of the apron has a thickness of.............................feet for a width of.............................feet.

20. Has the structure any weaknesses which are liable to cause its failure in high flows?...............No.........

....

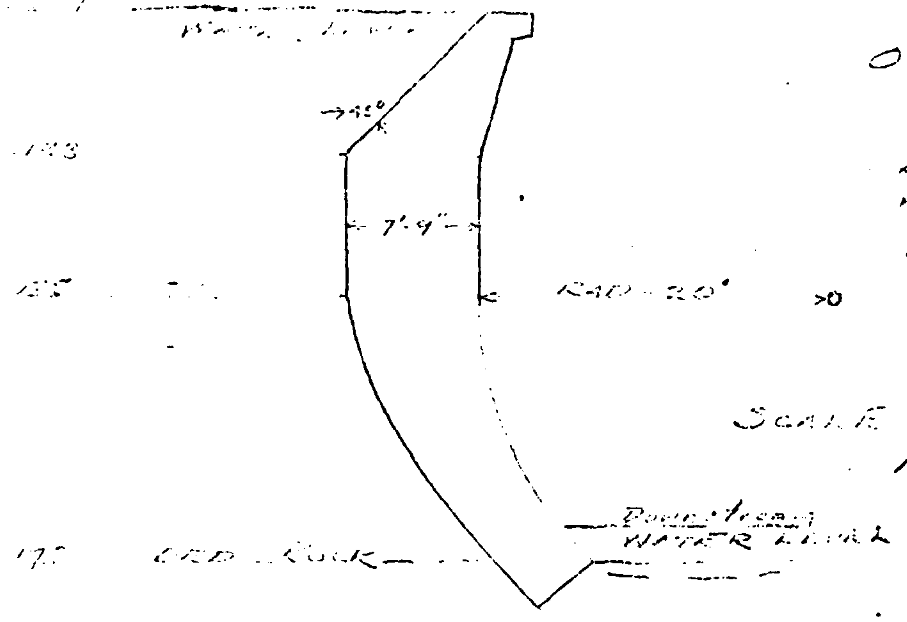
21. Sketches. On the back of this report make a sketch to scale for each different cross-section of the above structure at the greatest depth; giving the height and the depth from the surface of the foundation, the bottom width, the top width (for a concrete or masonry spillway at two feet below the crest), the elevation of the top in reference to the spillway crest, the length of the section, and the material of which the section is constructed; on the spillway section show a cross section of the apron, giving its width, thickness and material, and show the abutment or wash wall at the end of the spillway, giving its heights and thickness. Mark each section with a capital letter. Also sketch a plan; show the above sections by their top lines, giving the mark and the length of each; the openings by their horizontal dimensions; the abutments by their top width and top lengths from the upstream face of the spillway section; and outline the apron. Also sketch an elevation of each end of the structure with a cross section of the banks, giving the depth and width excavated into the banks.

22. Water Supply. The waters impounded by the above structure have (not) been used for a public water supply since.......1911.......by.......City of Ithaca, N.Y...

....

Section on ℄, at abutments ... = 8'6"

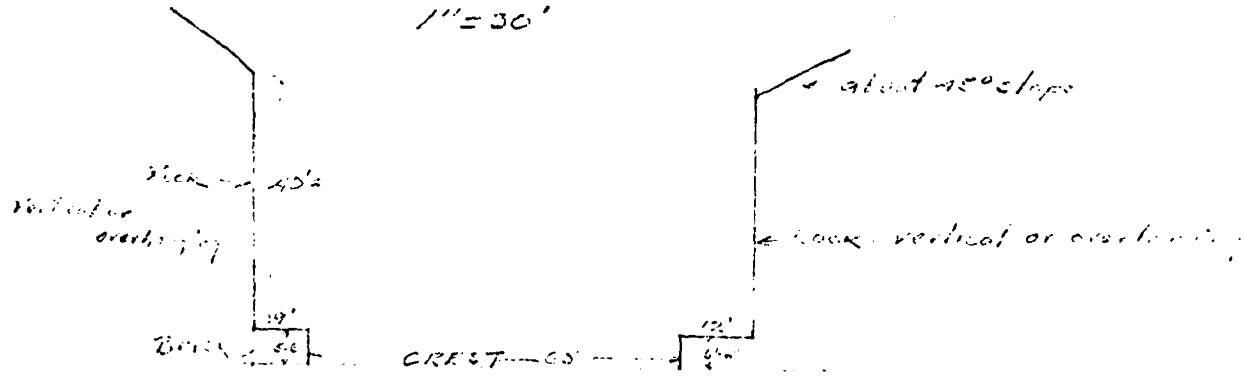
Otherwise similar



DAM IS CIRCULAR ARCH, 60' RADIUS WITH CENTRAL ...

SCALE OF SECTION 1" = 10'

PROFILE OF CREST AND BANKS
1" = 30'



DESCRIPTION GIVEN BY G. S. WILLIAMS IN TRANS. AM. SOC. C.E. V. 53 pg. 89

The above information is correct to the best of my knowledge and belief.

Ithaca, N.Y.
(Address of signer)

Carl Crandall
(Signature)
CITY ENGINEER
ITHACA, N.Y.
(A person signing for owner should indicate his title or authority)

Jan. 1, 1935
(Date)

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 981.

LAKE CHEESMAN DAM AND RESERVOIR.*

BY CHARLES L. HARRISON, M. AM. SOC. C. E., AND
SILAS H. WOODARD, ASSOC. M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. JOSEPH P. FRIZELL, BURR BASSELL, F. B. MALTBY, E. SHERMAN GOULD, E. W. HARRISON, FRANK C. HORN, CHARLES S. GOWEN, EDWARD WEGMANN, J. WALDO SMITH, E. KUICHLING, R. SHIRREFFS, GEORGE Y. WISNER, EDWIN DURYEA, JR., G. S. WILLIAMS, CHARLES L. HARRISON AND SILAS H. WOODARD.

PART I.

HISTORY, DESIGN AND CONSTRUCTION.

BY CHARLES L. HARRISON, M. AM. SOC. C. E.

The City of Denver, Colorado, with a population of about 150 000, is supplied with water by The Denver Union Water Company, a corporation which was formed in 1894 by the consolidation of the Citizens Water Company and the American Water-Works Company, both of which had previously been furnishing water to the city and to private consumers.

GENERAL CONDITIONS.

Denver is situated about 15 miles from the eastern foothills of the Rocky Mountains, in what is known as the semi-arid region of the West. The elevation of low water in the South Platte River, at

* Presented at the meeting of May 4th, 1904.

an arch dam was at an elevation of about 150 ft., where, on account of the span, a radius of about 200 ft. would be necessary. Limiting the stresses to those computed by the authors for the existing structure, it appears that an arch of about 25 ft. thickness would be required, and, starting with that, it would decrease to 1-ft. or any greater thickness that was desired, at the top, without increasing the stresses beyond those which the authors allowed in the structure as they designed it. Their stress, as they show it, is about 240 lb. per sq. in., or 35 000 lb. per sq. ft., and, taking that as a limiting stress, which, all will agree, is a perfectly safe stress for masonry of this character, a dam might have been designed for this place having a maximum thickness of 25 ft., and from that reducing to any desired thickness at the top; that design being a simple vertical cylindrical arch above the 150-ft. contour, and being of smaller radius below. Mr. Williams.

Another solution that appears in this case, is to have designed the dam as an inverted cone, and the spot would have been quite favorable for such a treatment. It will be recalled that the thrust of an arch under normal loads is equal to the pressure on the extrados into the radius of the extrados—not that of the center line, as is often incorrectly assumed—so that, by varying the radius, the thickness, or the total thrust to be taken by the arch, may be varied. Starting in that way, it would be possible to make the dam of equal strength, but much less than 25 ft. in thickness at the base, if such were desired. Whether or not it would be safe involves consideration of the permeability of the masonry, and that is possibly an open question with some.

Still another treatment would have been to make the base of the dam a segment of a sphere, and, recalling that the thrust in a spherical dome is only one-half of that in a cylinder of equal radius, still less material might thus be used in the base of the dam, and then, as the upward thrust of the sphere would have to be absorbed by the weight of the material above it, the radius of the sphere would be limited to that giving a thrust not greater than the weight of the material above its equator.

These are only offered as possible solutions, and there may have been reasons, other than structural ones, for not adopting such departures from former general practice.

Not long ago it fell to the speaker's lot to design a dam for a spot which seemed to be equally well, if not even better, suited for the arch solution of the problem, and, as illustrating a purely arch design, that location, the structure designed, and the structure built, are shown in Plates IX, X and XI. This site was in the vicinity of Ithaca, N. Y., about 2 miles from the center of the town, and the work was designed for the Ithaca Water-Works Company.

The Six-Mile Creek Dam.

Location and Conditions.—Six-Mile Creek, a stream having a quite precipitous drainage area of about 48 sq. miles above the point in

Mr. Williams. question, there passes in a northerly direction through a gorge or miniature cañon about 500 ft. long and 90 ft. wide. The location selected for the dam, Fig. 1, Plate IX, was near the upper end of this gorge, where the rock on the east side rises to a height of 90 ft. above the bed of the stream, overhanging in its rise 4 or 5 ft., and on the west side a similar wall, receding 6 ft. in its height, rises 70 ft. above the bed. On both sides, the rock was surmounted by a heavy deposit of drift clay, containing boulders, but quite impervious, and rising with a slope of nearly 30° for 50 or 100 ft. more. The rock was the bluish-gray shale, so common in that region, traversed at intervals of from a few inches to several feet with nearly parallel fissures, the sides of which, except near the exposed walls of the gorge, were in close contact, and, where open, the seams were filled with fine clay washed in from the covering beds. The planes of the fissures were also nearly parallel to the axis of the gorge. On the exposed faces the rock was weathered for a depth of about 6 in. to a varying extent, thereby showing very clearly its stratified character; but, where the weathered surface was removed, the faces of the fissures showed a smooth, dense rock without apparent horizontal seams, except at intervals, usually of several feet. The bottom of the gorge was covered to a depth of about 6 ft. by a deposit of sand and gravel, caused by the construction, a few years previously, of a small dam at its outlet. The bed itself was of shale rock, similarly fissured and nearly level throughout three-fourths of the width of the gorge, and rising in steps of about 4 ft. near the west wall.

As the location was only a short distance above the city, and, as a failure of the structure would involve considerable financial loss, not only to the citizens generally, but especially to the Water Company, whose pumping station was on the bank of the stream less than a mile below, it was at once apparent that a type of dam should be selected which would be stable against all possible contingencies.

The conditions being such as to call, first of all, for an overfall dam, and the seams in the bottom running longitudinally of the gorge and thus possibly permitting percolation and an upward pressure on the base, were to the speaker the strongest of arguments against the adoption of a gravity section of the ordinary type in this location.

Were the problem presented of carrying a roadway, even for the heaviest kind of traffic, across the gorge in question, no one would for a moment think of laying pipes or building culverts along the bottom and filling the chasm level full of masonry on top of them, nor would he put in a series of piers and connect them with short plate girders; but the one obvious and correct solution would be to span the depression with an arch either of metal, wood or masonry. Bearing in mind that the arch, under vertical moving loads, can never be in equi-

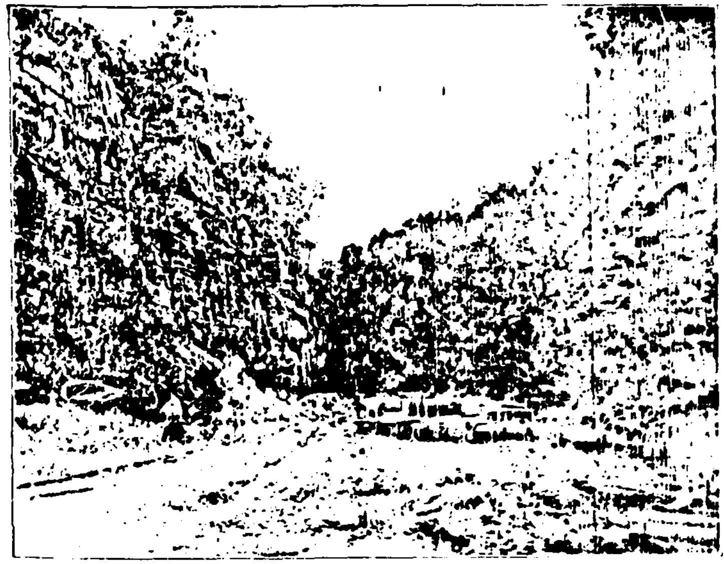


FIG. 1.—SITE OF SIX-MILE CREEK DAM.

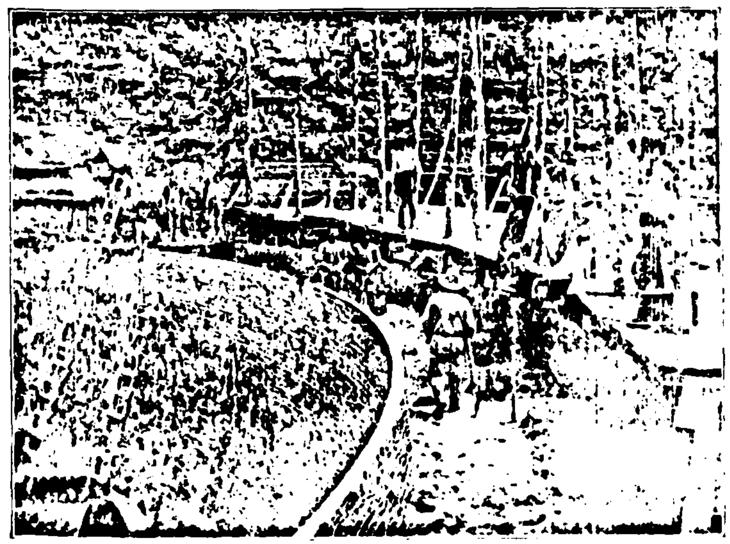


FIG. 2.—BUILDING THE SIX-MILE CREEK DAM.

librium, but must always resist varying bending moments, and that Mr. Willard. under normally applied uniform forces a circular rib will be in equilibrium, and subject to no bending moments, except those possibly induced by temperature changes and the compression of the material itself, which are also similarly possible in arch bridges, the propriety of applying the concrete arch to the problem becomes at once apparent, for it will be seen that the only possible means of failure for a circular arch under normal uniform forces is by the ultimate crushing of the material; and the conditions of the permeability of the base or foundation rock and consequent upward pressure underneath the dam, or a side pressure at the ends, have no influence upon the stability of such a structure in this location. The only possible ways for it to be destroyed by natural means are by the yielding of the abutments to such an extent as to cause the ultimate crushing strength of the material to be exceeded, or by the direct application of such a pressure as to bring about such a stress.

Design.—One of the chief criticisms directed against arch dams has been that, by reason of the rigidity of the base, the arch action could not be developed in their lower part, and, while the speaker is not one of those who would argue that a barrel is weaker against external pressure by reason of having the heads in it, yet, to overcome this objection, and avoid as far as possible stresses of opposite signs acting at right angles to each other, a condition which certainly weakens the material's ultimate capacity to resist either one, recourse was had to a design similar to that introduced in an egg-ended boiler, and the base, as shown by Fig. 30, was made of the form of a portion of a torus or ring. The whole structure was to be 90 ft. in height, and the radius of the vertical curve of the base was 20 ft., selected so that the upward thrust at Elevation 185 would never exceed the downward pressure transmitted through the material above. By this construction it became possible to utilize the bed of the stream as an abutment at *t s*, and still permit of elastic deformations and true arch or dome action near or at the base. By inclining the radius at the junction of the torus with the superimposed cylinder at *b r*, an up-stream thrust at this point was obtained from the former tending to oppose the pressure of the water and decrease the horizontal circumferential thrust in the cylinder. Similarly, the inclination of that portion of the structure above *b r* also introduces a thrust up stream, acting likewise to decrease the horizontal circumferential thrust. Above this plane, to Elevation 250.10, the section is made up of a series of frustums of conical shells. From 250.10 to 254.25 it is a segment of a ring, and from 254.25 to the crest at 260.0 it is a segment of a conical dome. The radii of the extrados or up-stream face are shown on the left of the section, and of the intrados on the right. The maximum radius of the extrados was 67.75 ft. and that of the crest 50 ft.

Mr. Williams. The axes of the two faces are not coincident, that of the down-stream face being 2.25 ft. up stream from that of the up-stream face, thus making the dam somewhat thicker at the abutments than at the center.

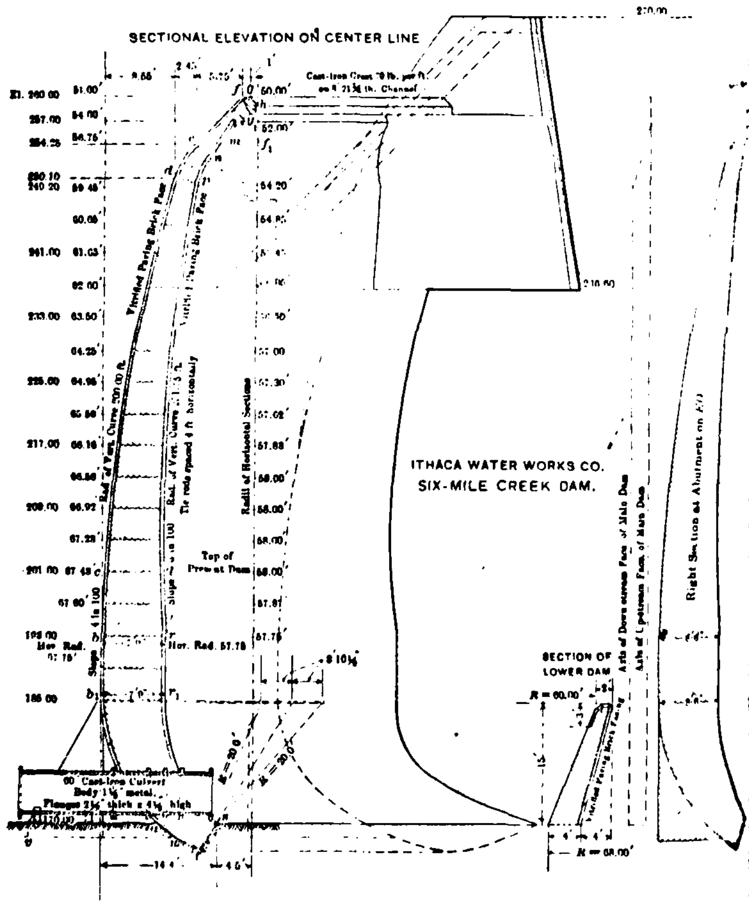


FIG. 80.

The shape of the crest was selected for the following reasons:

First, a form was desired which would discharge a maximum quantity of water at heads above 2 ft., and the one selected has been found by experiment to approximate closely to such a condition.

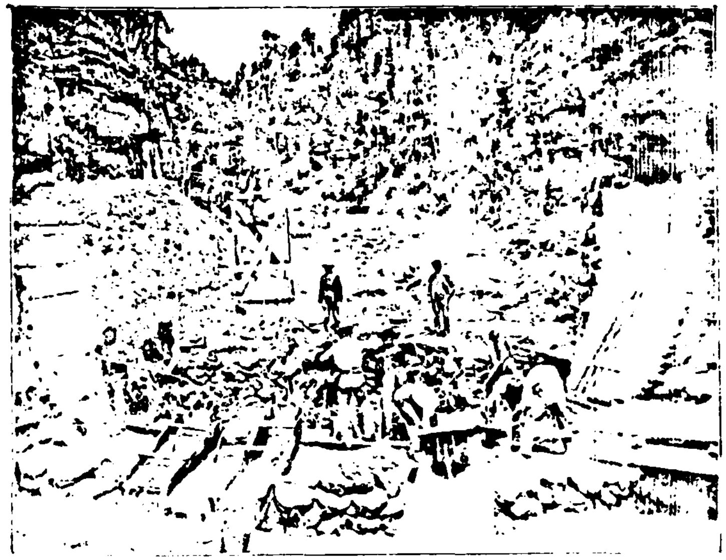


FIG. 1.—PREPARING THE FOUNDATIONS FOR THE SIX-MILE CREEK DAM.

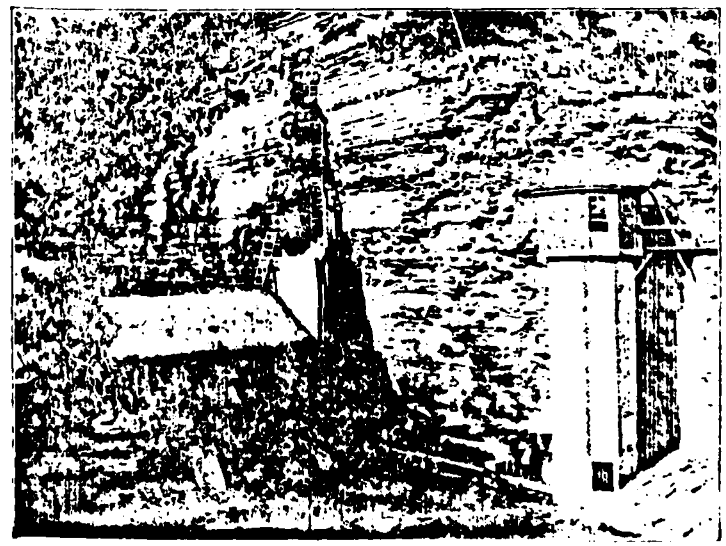


FIG. 2.—UP-STREAM VIEW OF EAST END OF SIX-MILE CREEK DAM

Second, a form was desired which would readily permit of ice climbing it, and the slope of 45° adopted answers this requirement well. Mr. Wil...

Third, a form was desired which would insure positive, certain and continuous aeration of the region behind the sheet, and the prevention of the formation of even a minute vacuum there, and the large space between the face of the dam and the falling water, in free communication with the air outside, *effectually precludes the occurrence of a* condition which, the speaker believes, has been, to no small extent, responsible for the failure of overfall dams in the past.

Fourth, a form was desired which would deliver the overfalling sheet well away from the toe of the section, and an inspection of Fig. 30 shows that this condition has been met.

As a further protection to the bottom, and also to insure a uniform upward thrust at $b\ r$, whether the pond were full or in flood, a second dam, 15 ft. high, was to be constructed about the middle of the gorge, 170 ft. down stream from the main dam, the overfall from which would be received in a pool formed by the old low dam already mentioned, which is 210 ft. farther down stream. This lower or middle dam was to be a segment of a frustum of a cone with a crest radius of 60 ft.

Computation of Stresses.—For preliminary purposes, the well-known formula, $T = p\ R$, wherein $T =$ the thrust or pull in the sheet, $p =$ the normal force, and $R =$ the radius of the face to which the force is applied, may be used, and, were the section cylindrical, p would be the water pressure and R the horizontal radius, and this formula would be rigidly applicable for the determination of the arch stresses. As, in the present design, the faces are generally inclined, this fact must be taken into account, and the formula becomes $T = p\ R \sec. i$, R being still the horizontal radius and i the angle of inclination of the face from the vertical. If the thickness be represented by F, then the unit thrust, $t = \frac{p\ R \sec. i}{F}$, for a section one unit high, omitting the effect of the inclination in producing a radial thrust opposite to T. As this counter thrust actually reduces T, it is evident that stresses computed by the foregoing formula will be greater than those really existing in the horizontal circumferential direction. For a flood 10 ft. in depth above the crest of the dam, which requires a run-off of 353 cu. ft. per sq. mile per sec., while the largest flood on record in this stream gave less than 100 cu. ft. and would require about 4 ft. head, the approximate thrusts in the horizontal arches by the above formula are as given in Table 9.

The thrusts in the torus base, being largely absorbed by the vertical arch of 240 in. radius, give much lower unit stresses.

For a final and more accurate determination of the stresses, the method used was as follows:

Mr. Williams.

TABLE 9.—APPROXIMATE STRESSES (IN EXCESS OF REAL STRESSES, EXCEPT ON OVERHANG NEAR CREST.)

Elevation.	Pressure of water, in pounds per square inch. p.	Horizontal radius, in inches. R.	$p\,R$ sec. i = thrust, in pounds. T.	Thickness, in inches. F.	Unit thrust in pounds per square inch t.
250	8.65	711	6 870	38	177
241	12.60	740	9 700	47.4	208
233	16.00	762	12 650	57	223
225	19.50	779	16 705	64.7	244
217	23.00	794	18 950	73	260
209	26.45	808	22 100	80	276
201	29.90	809	24 200	86	281
193	32.40	818	26 850	93	284

A vertical slice of the dam at the center, 1 in. thick at the up-stream face, was cut out by vertical radial planes and divided by planes normal to the up-stream face into 31 blocks, of which Nos. 1 to 7, inclusive, are on the overhang, 8 to 22 on the curved upper body, 23 is the cylinder, and 24 to 31 are on the torus base.

Beginning at the top, the force due to water pressure and that due to the weight of the block above the plane of its base are combined by a simple triangle of forces, Fig. 31, and the resultant, P, resolved into a horizontal component, P_h, and one normal to the base, P_n. For the next section, this resultant, P, is combined with the weight of the added block and the force due to pressure upon it, and a new resultant obtained which is resolved as above. Then, by Fig. 32, the forces acting on the block in question are:

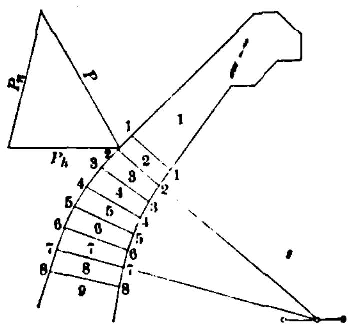


Fig. 31.

P_w = the water pressure on the face of the block acting normally thereto;

P_n = the component of the total pressure, P, normal to the base;

P_{n-1} = the component of the total pressure, P, normal to the plane of the top of the block, which = P_n for the section next above;

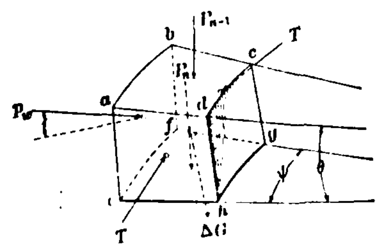


Fig. 32.

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$\varDelta G$ = the weight of the block;

T = the horizontal thrust in the arch ring;

R = the horizontal radius of the up-stream face;

θ = the angle which the top and bottom faces make with each other;

ψ = the angle which the side faces make with each other;

i = the angle which the normal to the up-stream face makes with the horizon.

Strictly, the slice should have been cut out between meridional planes, in which case its thickness, if 1 in. at the crest, would have been $\frac{67.75}{50.00} = 1.355$ in. at the cylinder; or, being 1 in. thick at the cylinder, it would have been 0.738 in. at the crest. The effect of this correction would be to reduce slightly the components of G, but this is compensated for by taking a low value for the weight of the material, 140 lb. per cu. ft., and by neglecting to consider the weight of the metal in the structure.* At the base of Section 7 the theoretical thickness for a slice 1 in. thick at the cylinder would be 0.874 in., and the thinness of the sections in a radial direction at the top makes the error possibly introduced of small practical moment.

For equilibrium, by Fig. 32:

$$p_w - \left\{ (P_n + P_{n-1}) \sin.\frac{\theta}{2} - \varDelta G \sin. i + 2\,T \sin.\frac{\psi}{2} \cos. i \right\} = 0,$$

or,

$$2\,T \sin.\frac{\psi}{2} \cos. i = p_w + \varDelta G \sin. i - (P_n + P_{n-1}) \sin.\frac{\theta}{2}$$

If H = the total horizontal force carried by the horizontal arch;

F_v = the area of the vertical faces;

F_h = the area of the normal faces;

t = the unit pressure, per square inch, on the horizontal arch at the center of the section;

s = the unit pressure, per square inch, on the vertical arch at the base of the section;

then

$$H = 2\,T \sin.\frac{\psi}{2} = \left[p_w - (P_n + P_{n-1}) \sin.\frac{\theta}{2}\right] \sec. i + \varDelta G \tan. i. \quad (3)$$

But, by the dimensions of the block, $\sin.\frac{\psi}{2} = \frac{1}{2}$ in. $\div R$, in inches;

therefore,

$$H = 2\,T \frac{\frac{1}{2}}{R} \quad (4)$$

or

$$T = H\,R \quad (5)$$

* The weight of the concrete alone, without the added boulders, was 141.4 lb. per cu. ft. The brick facing weighed 144.4 lb. per cu. ft. and the iron and steel averaged more than 0.8 lb. per cu. ft. of the entire mass, one-half of this weight being within 2 ft. of the crest.

Mr. Williams.

Mr. Williams.

TABLE 10.—ANALYSIS OF STRESSES IN SIX-MILE CREEK DAM, OMITTING INFLUENCE OF ATTACHMENTS AT SIDES AND BOTTOM. (Slide-Rule Computations.)

Section. No.	Section. Location.	Elevation: Feet above Cayuga Lake = City Datum. Base.	Elevation: Center.	Total Pressure above Base of Section, P, Pounds. Full pond.	Total Pressure above Base: 10-ft. flood.	Total Pressure Normal to Base of Section, P_n, Pounds. Full pond.	Total Pressure Normal: 10-ft. flood.	Water Pressure Normal to Face of Section, P_w, Pounds. Full pond.	Water Pressure Normal: 10-ft. flood.	Weight of section ΔG, Pounds
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	On overhang.	......	256.7	294	588	292	686	94.8	467	157.0
2	On overhang.	254.00	254.0	275	698	340	775	118.6	535	178.0
3	On overhang.	253.25	253.5	327	802	378	800	30.2	81	22.5
4	On overhang.	252.75	253.0	374	871	408	820	35.4	87.4	24.0
5	On overhang.	252.05	252.5	432	978	440	880	38.5	89	25.4
6	On overhang.	251.15	251.6	486	1 084	471	940	41.2	96	26.6
7	On overhang.	250.25	550.7	547	1 180	503	985	47.8	99	29.0
8	On overhang.	249.30	249.8	608	1 216	545	965	51.0	102	30.0
9	On curved upper body.	245.0	247.1	974	1 800	808	1 300	296.0	525	162.0
10	On curved upper body.	241.0	243.0	1 306	2 400	1 075	1 605	364.0	578	172.5
11	On curved upper body.	237.0	239.0	1 920	3 120	1 382	1 975	449.0	663	191.5
12	On curved upper body.	233.0	235.0	2 520	3 920	1 690	2 330	533.0	746	209.0
13	On curved upper body.	229.0	231.0	3 200	4 810	2 030	2 690	616.0	827	222.0
14	On curved upper body.	225.0	227.0	3 963	5 770	2 383	3 070	698.0	910	236.0
15	On curved upper body.	221.0	223.0	4 813	6 830	2 765	3 500	777.0	989	248.0
16	On curved upper body.	217.0	219.0	5 725	7 932	3 135	3 900	858.0	1 065	261.5
17	On curved upper body.	213.0	215.0	6 710	9 110	3 495	4 300	940.0	1 150	270.5
18	On curved upper body.	209.0	211.0	7 770	10 400	3 875	4 710	1 020.0	1 229	281.0
19	On curved upper body.	205.0	207.0	8 925	11 730	4 240	5 050	1 102.0	1 310	296.0
20	On curved upper body.	201.0	203.0	10 130	13 150	4 620	5 420	1 187.0	1 393	313.5
21	On curved upper body.	197.0	199.0	11 440	14 690	5 000	5 830	1 269.0	1 478	324.0
22	On curved upper body.	193.0	195.0	12 855	16 290	5 400	6 220	1 352.0	1 560	336.0
23	Cylinder	185.0	189.0	15 825	19 490	6 050	6 900	2 050.0	3 202†	692.0
24	On torus base.	182.8	183.9	16 620	20 480	6 290	7 050	833.0*	884†	158.0
25	On torus base.	180.7	181.8	17 400	21 300	6 300	7 160	844.0*	894†	158.0
26	On torus base.	178.6	179.7	18 140	22 120	6 330	7 230	856.0*	907†	155.7
27	On torus base.	176.6	177.4	18 880	22 890	6 350	7 260	866.0*	918†	153.6
28	On torus base.	174.8	75.7	19 530	23 540	6 330	7 230	875.0*	925†	150.0
29	On torus base.	173.1	174.0	20 120	24 200	6 200	7 160	885.0*	933†	148.5
30	On torus base.	171.5	172.3	20 600	24 800	5 950	7 120	888.0*	938†	134.1
31	On torus base.	170.0	170.7	21 050	23 350	5 690	6 880	893.0*	942†	121.6
										Σ 5 863.6

No.	Sine $\frac{\theta}{2}$	Sec. i.	Tan. i.	Radius of extrados of horizontal arch.§ R. Feet.	Areas. Vertical face of section. F_v. Square feet.	Areas. Base of section. F_b. Square inches.	Total Force Carried by Horizontal Arch. H. Pounds. Full pond.	Total Force: 10-ft. flood.	Thrusts in Arches, in Pounds per Square Inch. Horizontal. $t = \frac{HR}{12 F_v}$ ‡ Full pond.	Horizontal: 10-ft. flood.	Vertical. $s = \frac{P_n}{F_b}$ ¶ Full pond.	Vertical: 10-ft. flood.
	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)
1	0.000	1.414	1.000	54.40	13.45	26.2	291.0	817	98	275	11.1	26.2
2	0.000	1.414	1.000	54.70	15.26	26.9	315.0	934	103	279	12.8	24.8
3	0.487	1.345	0.898	56.95	1.93	28.2	36.7	76	90	187	13.4	28.4
4	0.487	1.230	0.713	57.60	2.06	29.8	14.5	28	33.7	65	14.5	27.5
5	0.487	1.161	0.600	58.15	2.18	31.1	0.7	23	1.5	51	14.1	32.0
6	0.487	1.108	0.414	58.60	2.28	32.1	10.9	20	23.3	43	14.7	33.3
7	0.487	1.068	0.306	59.00	2.48	32.8	9.3	15	18.4	30	15.8	31.0
8	0.010	1.046	0.277	59.80	2.57	33.2	30.3	54	58.3	104	16.4	29.1
9	0.010	1.040	0.270	60.05	14.20	40.7	352.7	566	117	199	19.9	32.0
10	0.010	1.035	0.250	61.15	15.25	45.9	379.2	611	127	204	23.4	35.0
11	0.010	1.030	0.237	62.12	16.95	50.1	482.4	691	147	211	27.6	39.4
12	0.010	1.027	0.228	63.05	18.55	55.3	563.5	770	160	218	30.6	42.1
13	0.010	1.022	0.200	63.87	19.85	57.8	696.5	839	170	225	35.1	46.5
14	0.010	1.017	0.171	64.60	21.10	62.3	705.3	908	180	231	38.3	49.3
15	0.010	1.014	0.150	65.32	22.45	65.7	774.9	972	188	237	42.0	53.3
16	0.010	1.011	0.130	65.87	22.90	69.2	841.0	1 039	202	248	45.3	56.4
17	0.010	1.007	0.110	66.36	24.68	72.8	909.8	1 106	204	249	48.2	59.5
18	0.010	1.005	0.089	66.74	25.40	76.3	975.0	1 171	213	257	50.8	61.7
19	0.010	1.003	0.065	67.08	26.70	79.6	1 043.8	1 234	218	258	53.3	63.5
20	0.010	1.002	0.044	67.33	28.36	81.6	1 113.8	1 304	220	258	56.7	66.8
21	0.010	1.001	0.044	67.52	29.50	85.0	1 188.8	1 380	226	263	58.8	68.6
22	0.010	1.001	0.044	67.67	29.50	87.6	1 264.3	1 455	241	278	61.5	71.0
23	0.0022	1.000	0.000	67.75	62.00	87.6	2 883.0	3 125	252	285	69.0	78.8
24	0.053	1.0016	0.0574	67.70	13.60	87.0	482.9	487	200	202	71.5	81.0
25	0.053	1.0135	0.1649	67.46	13.00	85.5	157.0	117	65	48.3	73.7	93.7
26	0.053	1.0374	0.2730	67.00	13.30	84.0	149.0	107	62	44.9	75.4	86.2
27	0.053	1.075	0.3940	66.28	13.10	81.6	164.0	116	69	48.9	77.8	89.0
28	0.053	1.128	0.5223	65.40	12.80	78.5	157.0	99	67	42.1	79.6	92.2
29	0.053	1.201	0.6657	64.32	12.10	74.3	175.0	110	78	48.8	83.6	96.3
30	0.053	1.300	0.8309	63.07	11.60	69.9	205.0	131	94	59.8	85.1	100.4
31	0.053	1.434	1.0282	61.60	10.70	64.4	259.0	160	124	75.7	88.3	108.0

* Component of water pressure on down-stream face subtracted.

§ Strictly, R, for center of gravity, should be used, since forces are combined there. By so doing the horizontal thrusts would be reduced about 2% at the top and 7½% at the cylinder.

† Component of water pressure on down-stream face, due to 8 ft. head on lower dam subtracted.

‡ At center of sections.

¶ At base of sections.

Mr. Williams.

and
$$t = \frac{T}{F} = \frac{H R}{F} \quad \text{.......................} \quad (6)$$

and
$$s = \frac{P_n}{F_b}. \quad \text{..........................} \quad (7)$$

For R and F, in feet, Equation 6 becomes
$$t = \frac{H R}{12 F} \quad \text{..........................} \quad (8)$$

Table 10 presents the elements of this computation for a full pond and for a 10-ft. flood.

Comparing the stresses induced for the two cases, Columns 20 to 23, the interesting fact is discovered that in this structure the unit stresses in the horizontal arches of the torus base—the weakest spot, apparently, if the design be judged by inspection simply—are less for the case of a flood than for that of a full pond, and, in spite of the apparently thin section at the toe, the maximum stress is only 124 lb. per sq. in., while the maximum anywhere in the structure, under assumed conditions far beyond any possible contingency, is less than 285 lb. per sq. in. Using the radius of the center of gravity of the cylindrical block, which is approximately 63 ft., the maximum unit stress in the dam is seen to be $\frac{63.00}{67.75} \times 285 = 265$ lb. per sq. in., or 19.08 tons per sq. ft.

Owing to the thicker section of the dam as it approaches the abutments, the corresponding maximum pressures on the rock are: for the east abutment, 247 lb. per sq. in., or 17.8 tons per sq. ft.; and, for the west abutment, 211 lb. per sq. in., or 15.2 tons per sq. ft. By way of comparison, it may be recalled that the pressures on the foundations of the Rookery Building, in Chicago, and those of the old Brooklyn Bridge are given as 400 lb. per sq. in., or 28.8 tons per sq. ft., while the concrete and low-grade rubble base of the Washington Monument is subjected to loads of 525 lb. per sq. in., or 37.8 tons per sq. ft. There are also a number of unreinforced concrete-arch bridges abroad which have been standing several years, in which stresses greater than 300 lb. per sq. in. either exist continuously or occur frequently, aside from those due to temperature changes and rib shortening.

The stresses near the crest are amply provided for by the crest casting and the steel channels in that portion of the structure.

Having now considered the conditions of full pond and flood, it remains to enquire as to the stresses in certain parts of the structure at low water, and when the pond is empty, should the latter condition ever occur after the completion of the work. Examining the horizontal thrusts due to the weight of the dam, it was found that for Sections 5 to 8, inclusive, the outward thrusts are decreasing downward, whence, in an ordinary dome, tension would occur in this region, and should in that case be resisted. Any yielding in the haunches,

however, must be accompanied by a lowering of the crest at the center, and, because of the rigidity of the abutments in this case, which prevents spreading along the chord of the dam, any lowering of the crest will be resisted by the hyperbolic arch formed along the vertical plane through the crest, and, consequently, the hoop usually supplied to a dome at the so-called joint of rupture is not needed here, although, to relieve the small tensions which might occur while the vertical arch resistance was developing, a hoop of 4 by ½-in. steel was provided at the haunch. Mr. Williams.

At the top of the torus base, similarly, tension would occur with the pond empty, were it not that a system of piers introduced under the heel of the dam acts as a support for the upper masonry at such times. These piers have no bond with the body of the dam, which is free to move away from them when loaded, but they act simply as wedges to keep the structure erect when there is no pressure on the back, and prevent tensile stresses at the top of the torus base.

As already stated, in the design of the Six-Mile Creek Dam, an attempt was made to eliminate, as far as possible, the influence of the beam or cantilever action which plays so extensive a part in the stresses of such curved dams as the one described by the authors, and the Zola and Sweetwater Dams, and even the more rationally designed Bear Valley Dam. To discover how successfully this has been accomplished, the midsection of the dam under a 10-ft. flood was subjected to an analysis similar to that presented for the Lake Cheesman Dam. A vertical slice of the dam, 1 ft. in thickness circumferentially, was taken, and, to simplify computations, rectified by projection upon a vertical tangent to the cylindrical portion, and this was divided into six sections 15 ft. in height. The average moment of inertia and the resulting deflections for each section were computed as for a beam, and then the average deflection of each section as an arch. The results are presented in Table 11, wherein, adopting the authors' notation:

a = height of section = 15 ft.;
D = the deflection;
I = the average moment of inertia for the section;
E = the modulus of elasticity;
X = the horizontal component of load on the section;
T = the thrust of the horizontal arch, the average value for a layer 1 ft. thick being used;
L = length of the arc of the arch;
A = area of arch layer 1 ft. thick;
α = one-quarter arc of arch before loading.

From Columns 18 and 19, the loads carried by the two systems of forces being inversely as the deflections under the same load, it is seen that at no point above Elevation 185, which is only 15 ft. above

Mr Williams.

TABLE 11.—COMPARISON OF ARCH AND BEAM STRESSES IN THE SIX-MILE CREEK DAM.

	FOR BEAM.								FOR ARCH.								PERCENTAGE OF LOAD CARRIED BY:	
	Elevations.		Thickness, in feet.															
Section.	Top.	Bottom.	Top.	Bottom.	Load, X, in pounds.	Moment of inertia, I.	$\frac{c^3}{6EI}$	ED for beam.	Elevation.	Radius, R, in feet.	Area of Ring 1 ft. thick, A, in square feet.	α	$L = \pi R \frac{4\alpha}{180°}$, in feet.	T, in pounds per square foot.	$ED = \frac{TL \cot. \alpha}{2A}$	$\frac{ED \text{ for Beam}}{ED \text{ for Arch}} = \frac{D \text{ for Beam.}}{D \text{ for Arch.}}$	Arch.	Beam.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
1	260	245	1.0	3.45	16 400	1.20	[illegible]	[illegible] 830 000	252.5	55.98	2.22	26° 48'	104.65	27 900	1 270 000	695	99.85	0.15
2	245	230	3.45	4.93	30 400	6.32	[illegible]	664 800 000	237.5	60.33	4.18	24° 07'	101.61	30 200	822 000	808	99.91	0.09
3	230	215	4.93	6.19	44 400	14.51	[illegible]	469 950 000	222.5	62.59	5.56	22° 59'	100.42	33 850	721 000	652	99.85	0.15
4	215	200	6.19	7.25	58 400	25.35	[illegible]	[illegible] 170 000	207.5	63.61	6.72	22° 31'	100.00	37 100	666 000	448	99.77	0.23
5	200	185	7.25	7.75	68 300	35.20	[illegible]	154 900 000	192.5	63.81	7.50	22° 25'	99.86	40 100	647 000	239	99.54	0.46
6	185	170	7.75	5.70	71 100	25.98	[illegible]	46 505 000	177.5	62.63	6.72	22° 56'	100.45	10 380	188 000	254	99.61	0.39

the base, does the beam carry one-half of 1% of the load, and it is also apparent that at half this distance above the base the beam cannot carry as much as 1 per cent.

Therefore, it may be concluded that the purpose of the designer, in this respect, has been accomplished, and that the stresses presented in Table 10 represent fairly the conditions in the structure.

At the west abutment, where, as already stated, the rock did not rise to the height of the crest, the thrust in the upper portion of the dam was taken up by a concrete abutment rising to Elevation 270, beyond which the dam was continued into the hillside as a series of elliptical half cylinders, the section of which at 45° was a circle, and which were inclined at 45° and cut off 10 ft. above the crest of the dam by a horizontal plane, the cylinders being supported by wedge-shaped piers under their springings.

As this part of the design has no bearing on the case of the Cheesman Dam, it will not be discussed further at this time.

Material and Construction.—The questions of material and construction have no particular bearing upon the Cheesman Dam, wherein the material was unquestionably the most desirable and best that could have been used, but, as considerable interest has been manifested in these matters, and as a description of the Six-Mile Creek Dam would be incomplete without them, they will be added here.

The body of the Six-Mile Creek Dam is of concrete composed of 1 part Alsen's imported Portland cement, 2 parts creek sand, 2 parts creek gravel and 2 parts broken stone from drift boulders, crushed to pass a 4-in. ring or less.

The voids in the sand amounted to about 42% of its volume. Mortar briquettes, 2 of sand to 1 of cement, 7 days old, indicated that it had a strength in tension equal to about two-thirds of that of standard sand.

The creek gravel was ordinary drift mixed with fragments of the shale rock of the region. Where the latter appeared as flat stones they were broken up or raked out.

The crushed stone contained about 15% of selected shale from the excavation in the rock walls, the remainder being field boulders. Flat stones were rejected both before and after crushing, as far as a reasonably close inspection discovered them.

The faces of the dam were of a single course of vitrified paving brick laid in a mortar of 1 part Alsen's cement, 1 part creek sand and 1 part crusher dust, and were anchored into the body by bent steel anchors, ½ by ⅛ by 7-in., turned up ½ in. at each end, placed at every fifth brick in every fifth course. On the up-stream face of the torus base the bricks were laid with the flat exposed, elsewhere with the edge exposed.

The brick used was that known as Catskill block, 3 by 4 by 9 in., a

Mr. Williams. very thoroughly vitrified shale brick, weighing 144.4 lb. per cu. ft. They were generally burned so highly as to be distorted considerably by the heat. Four samples, immersed in pails of water for four months, increased in weight less than one-tenth of 1%, and, when tested endwise in compression, failed by splitting lengthwise with a sharp report, at pressures varying from 2 300 to 4 600 lb. per sq. in.

Next inside the brick is a 3-in. mortar face of the same mixture as that used for the joints in the brickwork, which was laid at the same time as the concrete body, being separated therefrom by a plate of iron, until both were placed, when the iron was withdrawn and all were rammed together. Within this mortar face, and as close to the brick as convenient, *i. e.*, about 1 in. away, were set, above Elevation 185, bands of 3 by $\frac{3}{16}$-in. steel extending around the structure every 4 ft. in height, and united through the dam every 4 ft. horizontally by steel rods, $\frac{7}{8}$ in. in diameter, with a nut at each side of the bands. At Elevation 185, on the up-stream side, a band of 4 by $\frac{3}{4}$-in. steel was used and connected to the opposite 3 by $\frac{3}{16}$-in. band in a similar manner, to provide for possible tensions from pier to pier when the pond was empty. Over this steel skeleton, which was held in place by the horizontal rods extending into the brick faces, there was laid or hung a netting of crimped $\frac{3}{16}$-in. longitudinal and $\frac{1}{8}$-in. vertical wire of 4-in. mesh, extending from abutment to abutment on each face, and lapped one mesh and wired together at the horizontal joints of the sheets. All iron and steel was grouted carefully by dipping it in a trough as soon as it came on the work and before it had time to rust, and the bands and netting were placed as close to the outer faces as possible, their purpose being to distribute the stresses due to temperature changes and thereby prevent local cracks. The mortar and concrete were mixed in a Ransome mixer located about 150 ft. up stream from the dam, and the material was placed very wet. Into the body of the concrete were forced one-man stones as each layer was put in, they being carefully set with bed planes normal to the line of thrust, and were left projecting about half their height when a section was completed.

The brick walls were first laid up to a height of about 4 ft., No. 10 steel wires being bedded in every fifth course on the up-stream side, and, after setting about two days, the concrete was placed between them, they making the forms after Elevation 185 was reached. No deformation of the walls was detected in any part of the work, although braces on the up-stream side, used at first, were dispensed with entirely as the work progressed, and the down-stream side was left entirely unsupported all the time. Fig. 2, Plate IX, shows clearly the method of construction and the appearance of the work.

The foundations were carried down to sound rock, usually from 5 to 6 ft., but in one case, for a short distance, to 18 ft., below the bed

FIG. 1.—SECTION OF BASE OF SIX-MILE CREEK DAM.

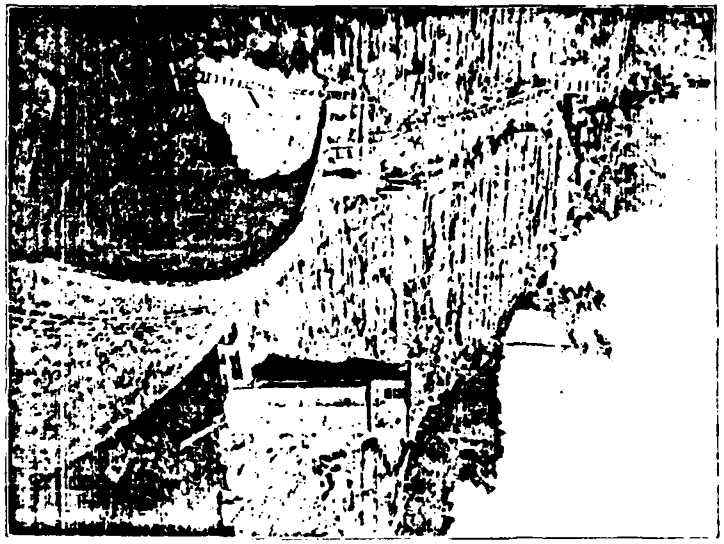


FIG. 2.—THE SIX-MILE CREEK DAM

of the stream. The longitudinal seams in the bottom were followed by 2 or 3-in. drill holes for from 4 to 6 ft. below the bottom of the foundations, and the holes filled with plastic clay well rammed. Fig. 1, Plate X, shows the end sections of the torus base completed, and excavation in progress for the middle. Mr. W.

Handling of the Stream.—As it was impossible to divert the stream from the gorge, it was necessary to provide for carrying it through the work, and the design contemplated the erection of the portions of the torus base at the abutments and in front of the piers, leaving the intermediate spaces open, but making all excavations, the stream being diverted from side to side during the work. Arches were to be sprung across the openings thus left, and the dam completed above Elevation 185, leaving passages through the base of sufficient capacity to deliver an ordinary flood. A permanent cast-iron culvert, 5 ft. in diameter, was also provided through the base and controlled by a gate. After the upper portions of the dam were completed, the openings through the base were to be filled, one at a time, at low water, the culvert then being able to carry the flow. Fig. 1, Plate XI, looking up stream, shows the base at the west end of the dam, and the low-water flow of the creek passing through the opening there while the center of the base is building.

The Dam as Built.—When it became noised abroad that a dam 90 ft. high and but 8 ft. thick at the base was to be built only two miles above Ithaca, to form a lake of 60 acres area, many people immediately saw visions of a Johnstown flood, and protests began to appear in the public prints. The plans, meanwhile, had been referred to four prominent members of this Society, the first of whom withdrew without making any report either favorable or unfavorable, and the other three, fully cognizant of all the conditions, including the action of the first, reported an unqualified approval. In due time bids were called for, and six bidders submitted proposals. Of these bidders, four were experienced engineers and contractors, three being members of this Society. Not one of them, after examination of the plans, specifications and location, expressed any doubt as to the stability of the structure. The tenders were received on unit prices for the several kinds of work involved, and the gross bids, exclusive of cement, which was furnished by the Water Company, based on the Engineer's estimates of quantities, were as follows: \$63 365; \$55 795; \$44 280; \$38 957; \$35 360, and \$34 488.

After work was begun, discussion of the structure continued, and a few so-called engineers, who had never seen the plans, expressed themselves in condemnation of the structure. Several others, after examining plans and location, expressed approval, and others still, perhaps more discreet than either, said nothing. The citizens of Ithaca invoked the aid of the State Engineer, the State Flood Com-

Mr. Williams.

TABLE 12.—DISTRIBUTION OF STRESSES AND ARCH THRUSTS IN BRICK-FACING ARCH.

Height 52 in., Thickness 4 in., Span 90 ft., Radius of Extrados 50.08 ft.

Position.	Deflections of Arch Crown.		Water pressure, in pounds per square inch.	Part of Total Load Carried by Arch and Arch Thrusts.									
				At crown.		At 4/10 span.*		At 3/10 span.		At 2/10 span.		At 1/10 span.	
	Arch carrying entire load.	Vertical beam carrying entire load.		Percentage of load.	Thrust, in pounds per square inch.	Percentage of load.	Thrust, in pounds per square inch.	Percentage of load.	Thrust, in pounds per square inch.	Percentage of load.	Thrust, in pounds per square inch.	Percentage of load.	Thrust, in pounds per square inch.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
At top	$\frac{78\,000}{E}$	$\frac{185\,500}{E}$	0.346	63.5	38	68.5	41	74.8	45	81.8	49	89.7	54
13 in. below top	$\frac{185\,000}{E}$	$\frac{94\,900}{E}$	0.82	33.8	48	38.9	55	45.8	65	56.2	80	71.9	102
At mid-height	$\frac{291\,000}{E}$	$\frac{51\,800}{E}$	1.28	15.12	34	18.18	41	22.9	51	30.8	69	47.2	106
13 in. above bottom	$\frac{398\,500}{E}$	$\frac{15\,950}{E}$	1.75	3.84	12	4.76	14	6.35	19	9.09	26	16.67	51
At bottom	$\frac{504\,000}{E}$	0	2.22	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0

* Fractional spans measured from abutment.

mission, and, finally, the State Health Commissioner, and injunction proceedings were threatened, but, apparently for lack of the necessary kind of engineering advice, were never brought. The breaking out of a typhoid epidemic, due to the water supply, but most probably not, as charged, to the infection of the water by work on this structure, raised public excitement to such a pitch that an almost unanimous vote in favor of municipal ownership was taken, although a similar proposition had been defeated less than a year before; and the State Health Commissioner announced at a mass meeting that his consulting engineers had declined to approve the design of the dam; but, he afterward stated, over his signature, that no official report had been made to him on the subject. Mr. Williams.

In consequence of all this, the company decided to stop the construction of the dam at a height of 30 ft., and, accordingly, it was built on the original lines to Elevation 193 and finished at Elevation 201, with a crest 60 ft. in radius overhanging on the down-stream side and having a 45° up-stream slope, as shown in broken lines in Fig. 30, and photographically in Fig. 2, Plate X, and Fig. 2, Plate XI, the former being a view of the up-stream face of the dam at the east end, and the latter the completed dam as it appears from the west abutment.

A flood of 3 ft. depth above the crest went over the dam on August 31st, 1903, before the runway used in its construction had been removed. By descending a ladder at each end of the dam, it was possible to look under the sheet and observe the face of the dam. A single leak, supplying a jet about half the size of a lead pencil, appeared about 8 ft. above the base. At this time the up-stream face of the dam, at the base, where the brick was laid up against forms, had not been pointed. Since pointing, neither this nor any other visible leaks have appeared.

A Test of the Brick Facing Arch.—During the construction, when the concrete was completed to Elevation 185 and the brick walls were up about 4 ft. higher, ready to receive the concrete filling, the work being in a similar condition to that shown by Fig. 2, Plate IX, the brick arch was subjected to an interesting test.

The brick walls, 4 in. thick, had been carried to a height of 52 in. above the top of the concrete on June 19th, 1903. The down-stream wall had all been laid within ten days, having been finished less than a week, and the up-stream wall laid within five days, a stretch of about 6 ft. near the east end, where the runway had been located, having been finished that afternoon. A heavy rain occurred the next day, and the water rose to such a height during the night as to overflow the dam to a depth of 0.8 ft. It began going over about 3 A. M., and was still overflowing at 9:30 A. M., the maximum height probably lasting for 2 or 3 hours.

Mr. Williams.

As the water rose it apparently dissolved the mortar in the short portion of the wall, about 6 ft. long, laid the day before, and broke through there before it went over the top, for the down-stream wall opposite this place was bulged, for about 10 ft., from nothing to about $1\frac{1}{2}$ in., the maximum being at about two-thirds of its height; and the up-stream brick wall, near the opening thus formed, was tipped over up stream, falling to the bottom of the pond above the dam, thus indicating that a powerful stream of water had recoiled from the lower wall against the concave side of the then unsupported upper wall, and wrecked it. At the far end of the dam, the up-stream wall fell over upon the concrete of the base, showing that the recoil had spent itself before reaching that part of the work.

In spite of having received what must have been a very severe shock, this brick arch, 52 in. high, 4 in. thick, and of 90 ft. clear span, with an up-stream radius of 58 ft. 1 in., stood throughout the flood, and, when the water subsided to a level with the top, no leaks appeared through the wall, except where openings had been left for the insertion of tie rods, and the wall was used, as originally intended, without any repairs or alterations.

TABLE 13.—Beam Stresses in Brick-Facing Arch.

Position from abutment.	Part of Total Pressure Carried by Beam 1 In. Wide.		Distance of center of pressure above base, in inches.	Moment at base, in inch-pounds.	Maximum Stresses on Horizontal Mortar Joints, in Pounds per Square Inch.		
	Percentage.	Pounds.			Tension.	Compression.	Shear.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Crown....	87.8	58.82	17.2	1 004	372	390	14.5
1/10 span....	85.2	56.69	16.9	962	357	365	14.2
2/10 span....	82.1	54.81	16.5	905	335	343	13.7
3/10 span....	77.2	51.56	15.8	814	301	309	12.9
4/10 span....	68.0	45.45	14.2	645	238	246	11.4

The stresses set up in this t[!] in arch have been examined by a process similar to that used by t[!] authors, and the results are presented in Tables 12 and 13. One point, not mentioned before, is to be noted, *viz.*, that, in considering a section of a dam as a beam, its action differs from that of an ordinary horizontal beam, since it does not begin to deflect until the load becomes sufficiently great to overcome the moment of the weight of the structure itself about some point in its base; and, if upon a yielding foundation, this point will not be the toe, as is usually incorrectly assumed in dealing with the overturning of gravity sections. Allowance for the weight of the

material has been made in the case of this brick arch, but was omitted in the analysis of the Six-Mile Creek Dam, and, apparently, in that of the Lake Cheesman Dam. In the former, by reason of its peculiar profile, it appears that the moment of the weight might become a considerable factor, and while not affecting greatly the deflections in the upper portions, might, to an appreciable degree, reduce that at the bottom, and hence the total deflections of the structure as a beam. Enquiring into this matter, the total weight of a slice of the dam 1 ft. wide is found to be, by Column 11, Table 10, $12 \times 5863.5 = 70\,362$ lb.; and, by inspection of the profile, it appears that the center of gravity will be somewhere within 5 ft. of the point of rotation at the base. The moment of the weight of the structure against overturning, therefore, is less than 350 000 ft.-lb. The total water pressure on the 1-ft. slice of the dam under a 10-ft. flood is $50 \times 62.4 \times 90 = 280\,800$ lb., and the point of application of its resultant is approximately 32.5 ft. above the base. The overturning moment, therefore, is $32.5 \times 280\,800 = 9\,126\,000$ ft-lb. The introduction of the weight factor into the computations of Table 11, consequently, would reduce the bending moment for the beam section less than $\frac{280\,800}{9\,126\,000} = 3.07\,\%$, an amount too small to affect to an important degree the results there set forth. Mr. Williams.

The values of the stresses given for the brick arch in Table 13 are computed on the usual assumption that the modulus of elasticity of the brick and the mortar joints is the same in compression as in tension. The tensions represent, not only the tensile strength of the mortar in itself, but also its adhesion to the brick surfaces, and the same is to be said for the shears. Tests by briquettes made of 2 parts of creek sand to 1 of cement, with 17% of water, when one week old, gave 270 to 300 lb. per sq. in. tensile breaking stress, and at two weeks 375 to 400 lb. The replacement of 1 part of the sand by 1 part of the crusher screenings probably increased the strength of the mortar somewhat, so that a tensile stress of 372 lb. is within the probable strength limit of the mortar ten days old.

It is noticeable, in such a long and slender arch as is represented by the brick wall, that the arch deflections at the crown would be much greater than the deflections as a beam, and the values for the portions of the loads taken by the two systems are to be looked upon as limiting approximations. The beam cannot be expected to carry more nor the arch less than these quantities. It is noticeable, however, that the thrust of the arch increases toward the abutments by reason of its deflection becoming less. As the radius, thickness and water pressure are unchanged from point to point, this increase can only be provided for or resisted by the absorption by the arch of some of the stress credited to the beam, or by the setting up of shearing

Mr. Williams. stresses of varying amounts, along horizontal planes. It follows that any complete analysis of the stresses in this arch, or any one in which the beam action is an important factor, must involve the consideration, not only of these, but also of secondary stresses set up by the action at right angles of primary stresses of opposite signs, all of which makes a very complicated problem.

In the Six-Mile Creek Dam no such uncertainty exists, as the beam action is practically eliminated, and the only secondary stresses to be considered are those due to rib shortening and temperature, the former of which cannot be nearly as serious as in the case of an arch bridge, with its necessary bending moments under both dead and live loads, and the latter of which is of less consequence than in such a structure, because the range of temperature is necessarily less.

It appears, then, that in the design of the Six-Mile Creek Dam there is presented a structure, which, for all practical considerations, acts wholly as an arch under uniform normal pressures, the equilibrium curve for which condition coincides with the center line of the section, and this seems to prove it to be as near an approach to the ideal as engineers are usually able to accomplish in structures of its magnitude.

The work was executed, under the speaker's personal supervision, by Messrs. Tucker and Vinton, Inc., of New York City, who used every endeavor to produce a most creditable structure. The speaker was assisted upon the construction by S. C. Hulse, Jun. Am. Soc. C. E., and Mr. Weston E. Fuller, whose care and interest in the work also merit commendation.

Mr. Harrison. C. L. Harrison, M. Am. Soc. C. E. (by letter).—Engineers who have attempted the solution of similar problems take a keen interest in the subject, as indicated by the number of discussions presented. The description of the design and execution of other works at different locations and under different conditions, which is really additional information on the general subject, is especially instructive in that it shows such a possible wide range of design and evinces a desire to improve on what has been accomplished in the past. Such a spirit leads to real progress.

It appears that some of the members who have discussed the paper considered it a presentation of the relative merits of the arch and the straight masonry dam, while the real object of the authors was to place before the Society an account of the design and construction of a dam at this particular location. Also, it does not appear to be fully understood that the dam was designed with a gravity section and was curved in plan because this form fitted the contour of the ground. It was believed the arch would give additional strength, but the attempt to determine the amount approximately was an after-consideration.

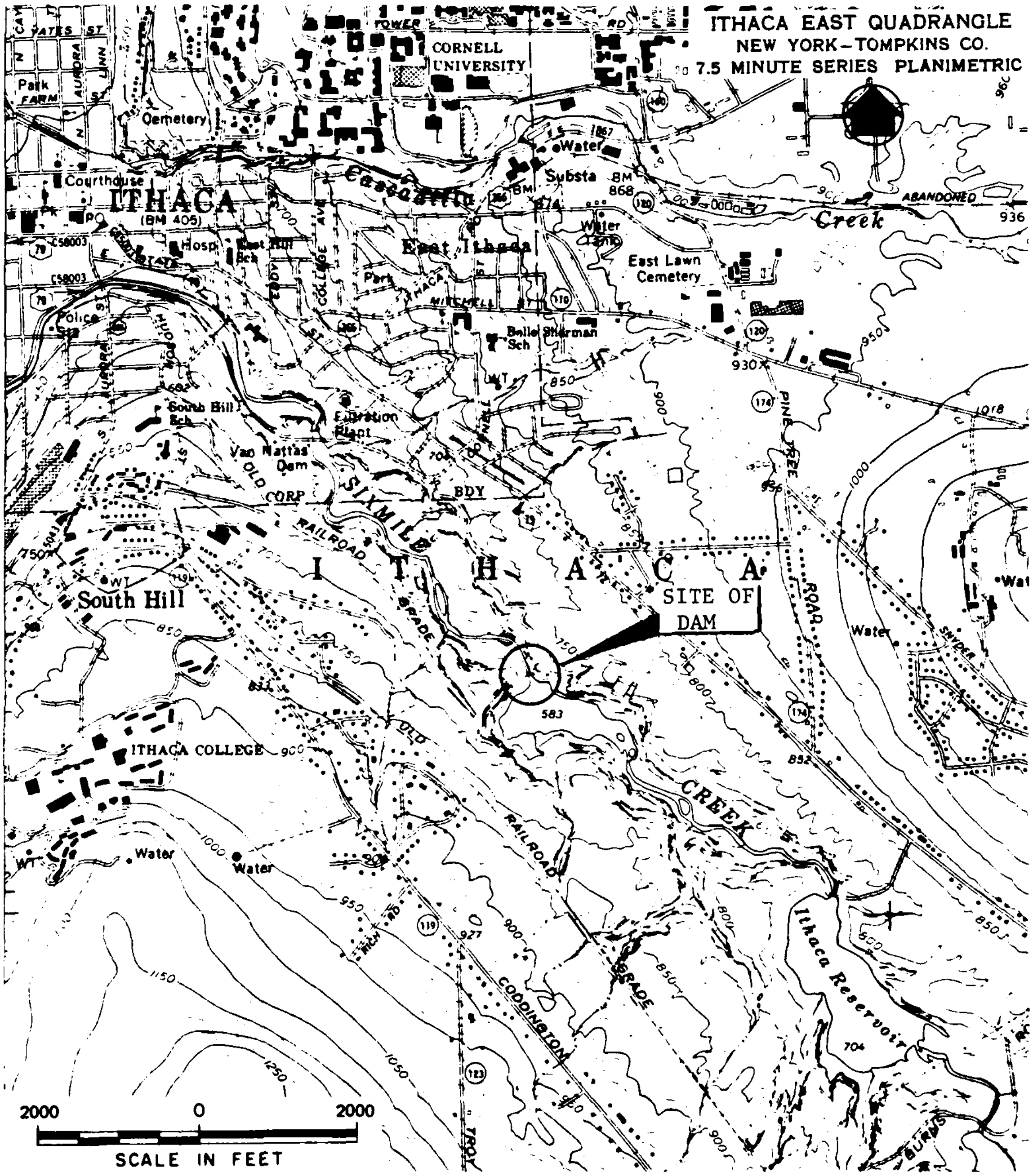
Omissions.—To present all the features in detail would require a

very long article, and perhaps befog the main question. A description Mr. Harrison. of the spillway and regulating devices was purposely omitted in the hope that A. E. Kastl, M. Am. Soc. C. E., the Chief Engineer, who completed the work, would take up these subjects in a discussion of the paper. The suggestion of Mr. Kuichling that more space should have been given to the hydrology of the catchment basin is a very worthy one, but this basin presents so many interesting features and is so markedly different from those of the Middle and Eastern States, that the subject is worthy of a separate and lengthy paper, which may be presented at some future time. However, it may now be noted that this catchment basin is wholly within the Rocky Mountains, at an elevation varying from 7 000 to 12 000 ft. above sea level. The precipitation is nearly all in the form of snow. These conditions make a flood during the winter season impossible. In fact it may be several months before the snow which falls in the deep gulches and on the northern sides of the mountains melts, and, in the meantime, a portion of it evaporates, and a considerable portion of it enters the ground, resulting in a lower and less rapid run-off than for water-sheds where the precipitation is in the form of rain.

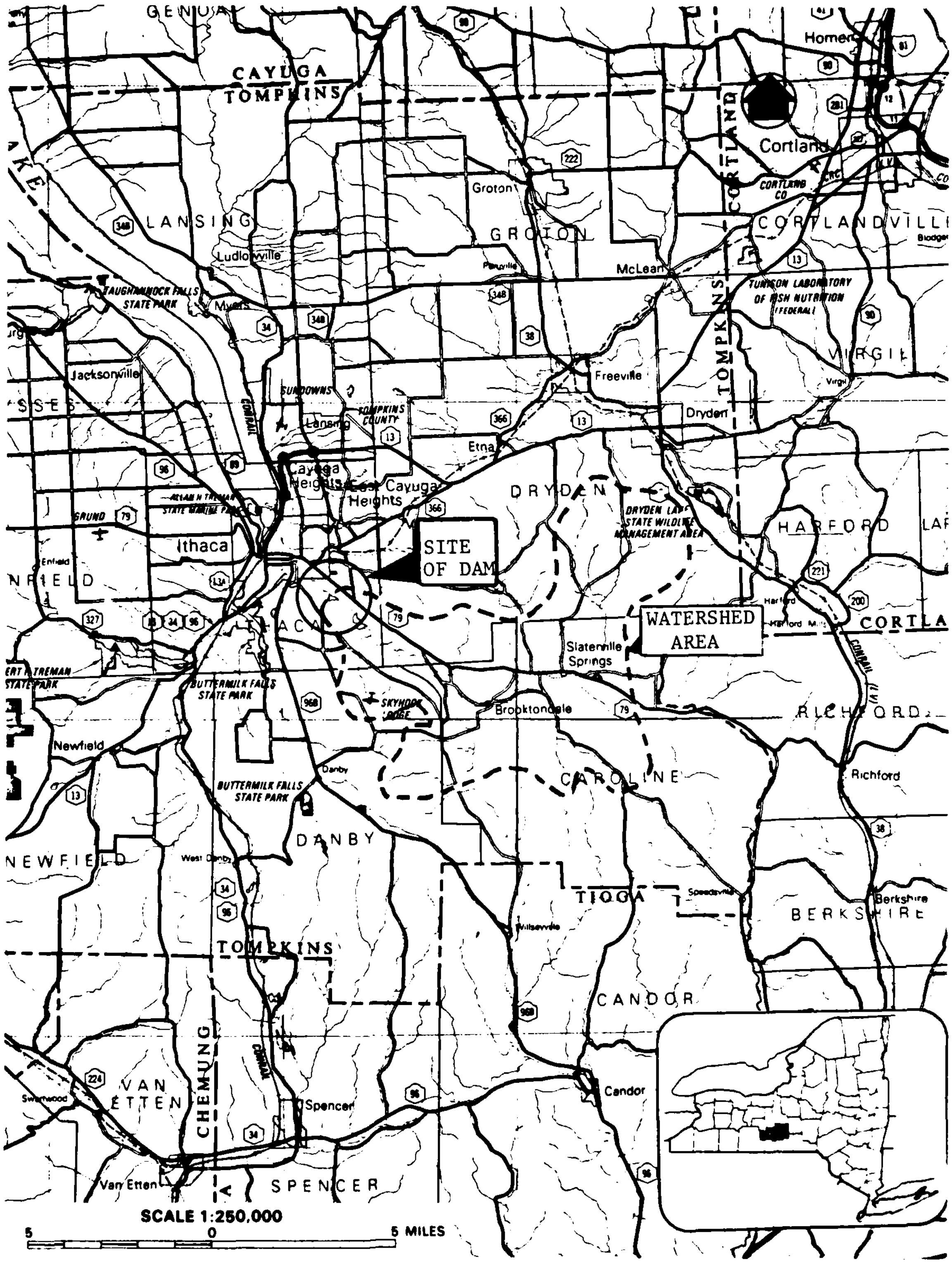
Alternate Designs.—By referring to Fig. 1, Plate IV, it will be seen that a curve of about 400 ft. radius is as small as could be used, and the location gives the shortest span across the cañon. The long chord or span of the dam as actually constructed is about 610 ft., measured between the ends of the center line of the top at Elevation 217 ft. The suggestion of Mr. Shirreffs, that a radius of 260 or 300 ft. might have been adopted with economy, could not possibly be followed, for the reason that the dam would not reach across the valley at that elevation. Likewise, the suggestion of Mr. Williams, that an arch dam of thin section might have been used, would probably not have been followed by him, as his suggested maximum span of 500 ft. for such a design is greatly exceeded at this location. It is probable that no engineer would build a high dam, depending wholly upon the arch for stability, with a radius greater than 300 ft. By referring to Plate IV, it will be seen that the bottom and sides of the cañon below Elevation 35 ft. (foundation of dam at approximately —10 ft.) are very irregular, and the width of the cañon is about one-fifth the base thickness of the dam. Between Elevation 35 and Elevation 90, the masonry is nearly in the shape of a keystone wedged in between the walls of the cañon. As pointed out by Mr. Wisner, who visited the work while in progress, this holds the lower 100 ft. of the masonry firmly in position and prevents any tendency to either overturn or slide. Mr. Wisner suggests that a base width of two-thirds of that used might have been adopted with safety, though the amount of masonry thus saved would not have been large. This is probably true, since the conditions make it practically impossible for the higher unit stresses

APPENDIX F

DRAWINGS



TOPOGRAPHIC MAP
SIXMILE CREEK DAM
I.D. NO. NY 395



VICINITY MAP
SIXMILE CREEK DAM
I.D. NO. NY 395